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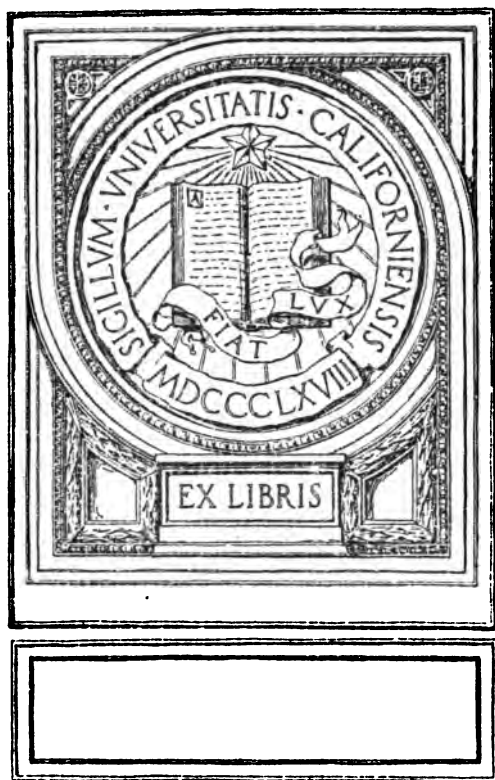
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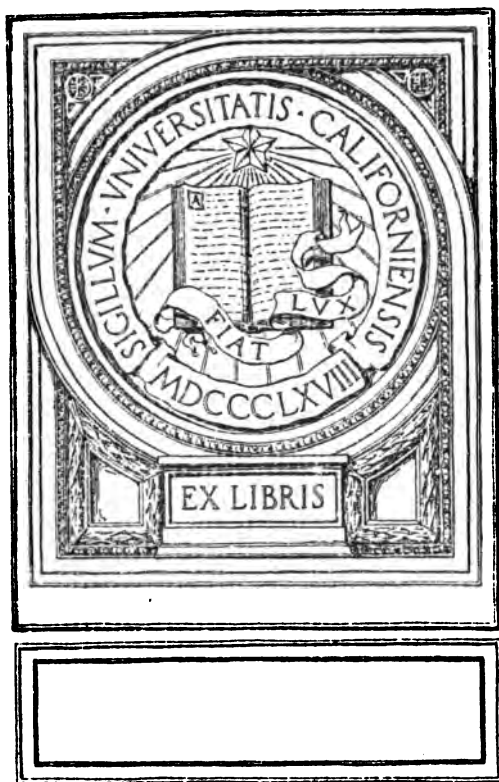
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**THE PRACTICAL DESIGN OF PLATE
GIRDER BRIDGES**

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THE PRACTICAL DESIGN OF PLATE GIRDER BRIDGES

BY

HAROLD HUGHES BIRD

ASSOCIATE MEMBER OF THE INSTITUTION OF CIVIL ENGINEERS
ASSOCIATE MEMBER OF THE INSTITUTION OF MECHANICAL ENGINEERS

WITH 2 PLATES AND 144 DIAGRAMS



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PREFACE

THIS book has been written for the benefit of young Engineers and Draughtsmen engaged on the design of Constructional Steelwork, and is particularly intended for those whose training has been more on the theoretical than the practical side.

It is the result of practical experience both in manufacture and design, and the examples given and opinions expressed are based on actual practice.

In writing this book, it has been assumed that the reader has a certain knowledge of Applied Mechanics and the Strength of Materials, and beyond mentioning the case of a Simple Beam, these subjects have, in general, been considered outside the scope of the present work, and the reader is asked to refer to text-books on the subjects.

The Author trusts that this work will assist young Engineers over the gulf between the knowledge of the principles of design and the actual application of that knowledge to the design of Plate Girder Bridges and Constructional Steelwork generally.

In the preparation of several chapters, assistance has been received from Mr. H. H. Vaughan, B.Sc. (Glasgow), Assistant Professor of Engineering, Heriot-Watt College, Edinburgh, to whom the Author expresses his indebtedness and thanks.

HAROLD H. BIRD.

CARDIFF, 1920.

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THE PRACTICAL DESIGN OF PLATE GIRDER BRIDGES

CHAPTER I

ADVANTAGES OF PLATE GIRDER BRIDGES

It is now generally recognised that a steel plate girder bridge is a most economical and efficient type of structure for spans ranging from 15 to 80 ft., carrying roads or railways.

The greater number of steel bridges designed are of this type, a type evidently possessing advantages justifying its adoption (now almost a habit), to such a large extent.

In the first place, plate girders are so simple that there is little possibility of error, in the calculation of either stresses or design, and the cost of manufacture is considerably less than any other kind of construction, while the risks of defects due to faulty material and workmanship are reduced to a minimum. They can be transported with little difficulty, oftentimes in one piece, and erected at site in a short time without great trouble, and at a small cost. When erected they require little attention beyond occasional painting, the only maintenance necessary for a properly designed bridge.

Plate girder bridges possess several advantages compared with brick or masonry arches. Not only is a much greater headway required to give the necessary rise to an arch, but the total height required for construction is considerably greater than in the case of a steel girder superstructure, both important considerations, in situations where roads have to be raised or lowered to secure the necessary headway. This usually involves the construction of long approaches with high banks or deep cuttings, which greatly add to the cost of the work.

In colliery districts where the surface of the ground is liable to subsidence due to underground workings, an arched bridge is a most undesirable structure.

If subsidence should take place a brick or masonry arch would most likely be damaged beyond repair ; but with a girder superstructure the effect

of settlement can usually be remedied by packing up the girders to the desired height.

The joint qualities of simplicity, cheapness, endurance, and low maintenance costs are combined to such an extent in plate girder bridges that their adoption for moderate spans has become the standard practice of most Engineers.

PRINCIPAL CONDITIONS TO BE OBSERVED IN DESIGNING

The design of steel bridgework comprises a most important branch of civil engineering, and much care and foresight must be exercised by Engineers when called upon to design such structures.

There are three principal conditions to be observed in designing steel bridgework of any description, viz. :—

1. The structure must be strong enough to carry the heaviest loads, including its own dead weight, under the worst possible conditions of loading.

2. The material must be so proportioned that there shall be a minimum of internal stress and strain, and a maximum resistance to the combined dead and live loads, with a minimum cost of material and workmanship.

3. Due provision must be made for future maintenance, and designs should allow of easy access for painting and inspecting all parts liable to corrosion.

The importance of the first condition is patent to all designers of steel-work, and is fully dealt with in Chapters III. and V.

In the second condition, theory and practice go hand in hand. From a theoretical point each part of the structure should be so proportioned that in the event of failure all parts should fail simultaneously. Of course, this is a condition that never occurs in practice. It has been said with a certain amount of truth, that no man living can accurately determine the stresses in a plate girder.

In plate girders, as in other types of girders, there are secondary stresses of which we have only an empirical knowledge, but by basing calculations on well-established deductions it is quite possible to produce a structure both scientific in design (so far as this is possible) and efficient in practice. Two separate men, one a theoretical, and the other a practical man, having the same bridge to design regarding span, loading, and working stresses, may produce entirely different designs. On the one hand, the bridge designed by the theoretical man may have 10 per cent. less material, be more scientifically proportioned and of better appearance than the one designed by the practical man, which latter, on the other hand, may cost 10 per cent.

less than the other, and at the same time be quite as efficient. The reason for this difference is not far to seek.

In the former case, the designer, by conforming strictly to theoretical requirements and seeking after architectural effects, has produced a design requiring, perhaps, double the amount of workmanship in manufacture than the latter one. The one principle underlying economical designing is to make workmanship an absolute minimum for the weight of material required.

The third condition stated above, although often lost sight of by designers, is of great importance. As long as a railway or road is open and traffic running, wear and tear of bridges, etc., takes place, and maintenance becomes a necessary duty. In reckoning the real cost of any structure, not only must the capital expenditure and the interest thereon be considered, but also the cost of future maintenance, consequently, the bridge requiring the least capital outlay, and a minimum maintenance, will be, other things being equal, the most economical and efficient.

A little foresight and extra expenditure in the first instance, especially in trivial matters, apt to be overlooked, may be the means of saving maintenance costs, which in the aggregate, will be considerable.

Badly designed bridges which cannot be maintained with efficiency and economy, soon show signs of weakness, corrosion sets in with serious results, and in the course of time, after a very short life, reconstruction becomes necessary, accompanied as it usually is by dislocation of traffic, inconvenience and expense.

In designing steel bridgework, every endeavour should be made to produce structures as simple as possible consistent with strength and utility, always bearing in mind that good designing consists not only in providing for immediate requirements, but also for the after maintenance, a condition upon which the efficiency of any structure largely depends.

Although the above conditions generally apply to all types of bridges, there are perhaps none which require to comply with them more, or where such careful calculation and consideration are required, than bridges carrying a railway, exposed as they are to all sorts of weather and atmospheric conditions, as well as to extremely variable live loads.

The truth of this statement will be realised, when it is considered that the greater proportion of bridge building on British railways consists of the renewal of old cast-iron and wrought-iron structures, which are incapable of safely withstanding the heavy express traffic of the present day, owing either to the great increase of axle loads since they were built, bad design, or corrosion; or any two, or perhaps all three of these causes may have been combined.

PRINCIPAL CONSIDERATIONS AFFECTING DESIGNS

In Great Britain all bridges over and under railways must be designed in conformity with certain regulations issued by the Board of Trade, particulars of which are given in the Appendix.

In addition, the design is also governed by the limitations of the maximum structure gauge of the particular railway, and oftentimes by the requirements of local and other authorities. These requirements principally apply to strength, width, headway, and sometimes appearance, but in the case of bridges abroad they need not be considered.

Little difficulty is experienced in designing the main girders of a bridge, as the depth is usually made to some proportion of the span, and within certain limits may be varied, but the type and design of the floor system depends upon the available depth of construction, which is the distance from the rail or road level carried by the bridge to the lowest part of the underside, or soffit of the bridge, and in most cases, especially of reconstruction, this dimension is usually very shallow, and cannot be increased except by raising or lowering the rail or road level, or by allowing the structure to encroach on the headway underneath.

The Railway Clauses Act, 1845, fixes the minimum headway to be provided and maintained with regard to bridges for roads under railways; also Acts of Parliament, authorising the construction of railways, frequently fix the minimum headway, and the authorities concerned are very careful to see that no encroachment on the fixed headway takes place.

Many old bridges appear to have been badly designed in the light of present-day standards, and are invariably found to have very shallow floors, and it is a common occurrence to find girders one-twentieth to one-thirtieth of the span in depth, so that when reconstruction becomes necessary, many very troublesome points arise in designing the new floor system.

If the available depth of construction of a railway underbridge is very shallow, and the rails are level or on a slight gradient, it is little trouble to give them a lift, but frequently the gradient is too severe to allow any raising, and it may happen that the bridge is near to some station, where any lifting of the rails would, perhaps, not only mean making the gradient stiffer, but also the raising of the station platforms and the floors of the station buildings, the cost of which would prohibit any increase in the construction depth of the bridge.

Another troublesome case that may occur in this respect is that where a bridge is situated in close proximity to sidings, when any raising of the rails

would interfere with the existing siding connections, and prove very costly and complicated.

Under circumstances such as these, the floor system of a new bridge must be designed to suit the existing construction.

This question of insufficient construction depth is one that presents itself in many reconstructions of railway underbridges.

In the case of bridges on a new line of railway, ample construction depth is usually available, consequently the difficulties experienced in the renewal of bridges are to a great extent obviated. The same remark applies to bridges in undeveloped countries, where the Engineer has a free hand to design structures, unrestricted by questions of headway and shallow construction depth.

An important consideration influencing the design of railway underbridges, is the question of erection and renewal in regard to the exigencies of present and prospective traffic. The renewal of such bridges involves interference with the running lines, in many cases with two lines of way at the same time, an operation attended with grave risks, and accompanied more or less, by delay to traffic, resulting in considerable inconvenience and expense.

In deciding upon the type of bridge to be adopted, careful consideration should be given to this question.

CHAPTER II

TYPES OF PLATE GIRDER BRIDGES

UNDER this head it is proposed to consider the various types of plate girder bridges together with the different floor systems.

All girder bridges may be divided into two main classes, viz. :—

“ DECK ” BRIDGES AND “ THROUGH ” BRIDGES

A “ Deck ” bridge is one in which the floor is carried on the top flanges of the main girders, as shown in Fig. 8.

A “ Through ” bridge is one in which the floor is carried on the bottom flanges of the main girders, as shown in Fig. 1.

The adoption of either of the above types depends principally upon the headway to be given, and the available depth for construction.

“ Deck ” bridges are the more economical, and are usually adopted in cases where headroom is not limited, as, for example, where a road or railway crosses a river or valley.

“ Through ” bridges are adopted in cases where the available depth for construction will not admit of the “ Deck ” design.

A bridge carrying a railway over a road, or a road over a railway, must be built with a certain amount of headway under the bridge, *i.e.* the distance from road or rail level to the underside of the main girders, therefore, to avoid high banks, approaches, etc., to the bridge, the railway or road is carried as low as possible and a “ through ” bridge adopted.

Railway bridges are classified as overbridges when carrying roads, etc., over the railway, and underbridges when carrying the railway over roads, rivers, etc., and these are still further classified as public road and accommodation bridges, according to the description of roads they carry or cross.

FLOOR SYSTEMS OF RAILWAY UNDERBRIDGES

Broadly speaking, there are two classes of floor systems, viz. :—

1. Free floors, in which the permanent way is of the ordinary cross-sleeper road, embedded in ballast continuous over the bridge, and

2. Tied floors, in which the rails are attached to timber beams laid longitudinally over the bridge, these latter being bolted to the steelwork of the bridge.

From a maintenance standpoint it is desirable to have the permanent way over bridges uniform with the standard type of the line, but there are cases where the ordinary cross-sleeper road is out of the question, and others where a longitudinal-sleeper road is precluded.

In bridges of long span, where the proportion of dead load is very great compared with the live load, it is essential to reduce the dead weight as much as possible, and in such bridges the adoption of the ordinary ballasted cross-sleeper road is rendered too costly owing to the great weight of the ballast, so resort must be had to some kind of tied floor, with the rails carried over the bridge on longitudinal timbers, or way-beams, a much lighter, and in this instance, a cheaper road than the cross-sleeper type. In bridges of small span, such as are being considered, both kinds of floors are used, but a free floor has many advantages over a tied floor, providing there is ample construction depth for its adoption. If the latter system is adopted, and the rails carried over the bridge between continuous angle fish-plates, the longitudinal timbers have to be shaped on both their lower and upper surfaces to suit the camber of the bridge, and to give the necessary cant to the rails; should the bridge be situated on a curve the timbers would require very awkward cutting and packing to give the required super-elevation and cant, rendering relaying or raising of the road complicated and costly. A further disadvantage of tied floors is their rigidity, as the timbers carrying the rails, being bolted to the floor of the bridge, are thus prevented from creeping with the rest of the track.

Every endeavour should be made to adopt designs where the ordinary cross-sleeper road can be used, a road that will save much trouble and cost in maintenance compared with the longitudinal type.

All railway-engineers familiar with the maintenance of steel bridges are aware from experience that no part of the structure causes more trouble in maintenance—oftentimes rendering reconstruction necessary—than the floor system. The early railway-engineers appear to have given more thought to the design of the main girders than to the flooring, as in many instances where reconstruction is necessary, owing to the latter being too weak to bear the heavy loads of modern traffic, it is found that the main girders are amply strong. At the present day the wheels of locomotives are closely spaced, axle loads are far heavier than hitherto known, and special vehicles are constructed for carrying heavy freight, and these have the effect of putting very severe concentrated loads upon the details of the floor system,

so it behoves Engineers to give careful consideration and thought to the design of all the floor details, bearing in mind, that in the majority of bridges, these are the parts of the structure first affected by the live load before it is transmitted to the main girders further removed from the load.

TYPES OF RAILWAY UNDERBRIDGES AND FLOOR SYSTEMS

Advantages and Disadvantages

Timber Floors.—Many bridges have been built with timber floors, but such floors are seldom adopted nowadays. They have the advantage of being cheap, light, elastic, easily renewed, and if made of good material are very durable, but they have the disadvantage of retaining moisture for a considerable time, thereby hastening corrosion of the metal with which the timber is in contact. Cases can frequently be seen where the wrought-iron main girder webs of bridges are quite rusted through at places where the timber floor abuts against them, due, no doubt, to the holding of a film of water between the timber flooring and the girder web.

Another serious objection to the use of timber is its inflammability, as there is always the danger of fire being liable to be caused by hot ashes falling from the fireboxes of the engines.

Modern Bridge Floors. Free Floors.—Fig. 1 shows diagrammatically a cross-section of a three-girder "through" bridge, with cross-girders, rail bearers, and plated floor, a type very commonly used, and if carefully designed leaving little to be desired; it is also, perhaps, the most economical for spans up to about 60 ft. as a maximum, as above this span the depth of the centre girder, unless it were made very shallow and therefore uneconomic, would encroach on the clearance specified by the Board of Trade, which states that: "no standing work (other than a passenger platform) is to be nearer to the side of the widest carriage in use on the line than 2 ft. 4 ins., and this at any point between the level of 2 ft. 6 ins. above the rails, and the level of the upper parts of the highest carriage doors." The centre girder having practically double the load to carry compared with the outer girders, must be proportionately stronger. The minimum construction depth for bridges of this type is about 2 ft. 10 ins., made up as follows:—

						ft.	ins.
Depth of cross-girders at centre	...	...	...	...	...	1	4
Asphalt	...	...	...	...	...	...	1
Ballast	...	...	...	...	...	...	4
Rail, Chair, and Sleeper	...	...	...	...	...	1	1
Total	...	...	...	...	...	2	10

By reducing the depth of the cross-girders to 1 ft. 2 ins., and allowing 2 ins. of ballast under the sleepers, it is possible to design a bridge of this type with a minimum construction depth of 2 ft. 6 ins., but such shallow cross-girders are apt to prove troublesome to design, and 2 ins. of ballast under the sleepers is rather a scanty amount, and cannot be recommended.

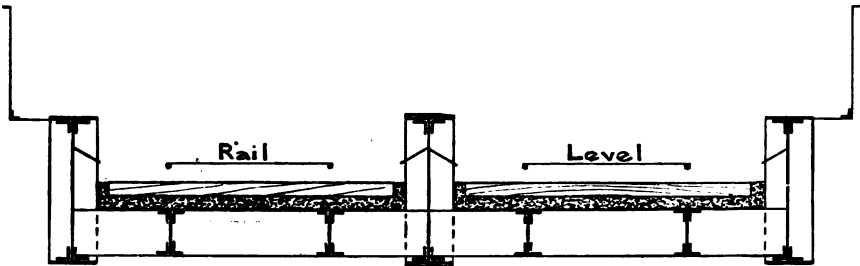


FIG. 1.

In cases where the construction depth is very shallow, the design shown in Fig. 2 is sometimes adopted. It is very similar to the previous one, the difference being that the rail-bearers are made shallower, and the plated floor suspended below the tops of the cross-girders and attached to the top flange of the rail-bearers. The sleepers are spaced between the cross-girders, the rails being suspended over the cross-girders just clear of the steelwork. This

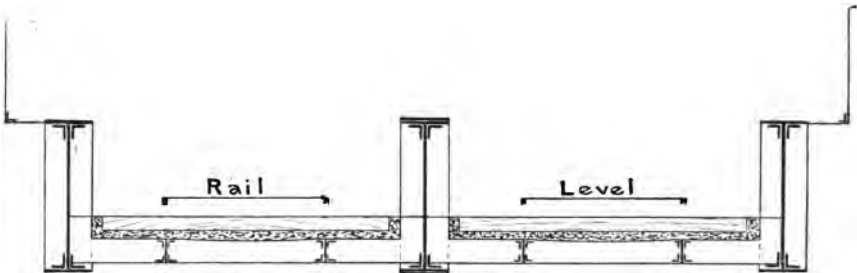


FIG. 2.

type of floor has the advantage of allowing a cross-sleeper ballasted road to be used with a small construction depth, but it is very objectionable in maintenance on account of the great difficulty experienced in properly draining the various bays, and also because of the little room left for packing the sleepers. It can be used for a construction depth of 2 ft., but it is a very undesirable type, and should be avoided if possible. Fig. 3 shows a "through" bridge with two central girders and two outside girders, each

line being carried by separate and complete spans. This type can be used for spans up to 36 ft., without widening the 6 ft. space, and where the depth of the centre girders is such, that the height from rail level to the top flange, is under 2 ft. 6 ins., and the horizontal clear distance from rail to flange is not under 2 ft. 2 ins. It can, however, be used for spans up to 60 ft., but this would probably involve widening out the 6 ft. space, so as to provide room

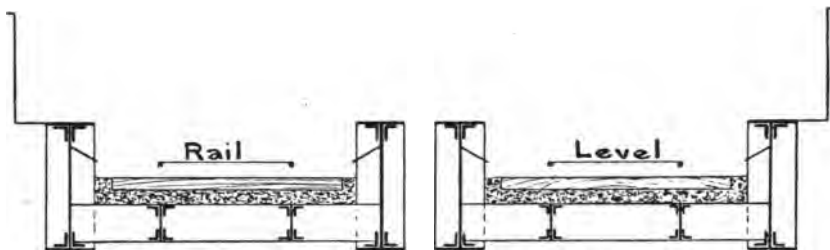


FIG. 3

for the two central girders to clear the structure gauge, with ample space between them to allow of access for painting. This type of bridge is eminently suitable where the exigencies of present and prospective traffic render renewal and erection a difficult operation, and also in districts where subsidence is liable to take place, on account of underground workings. The weight of the steelwork will be a little heavier than in the "three-girder" bridge.

Another type of "through" bridge is shown in Fig. 4. This design is

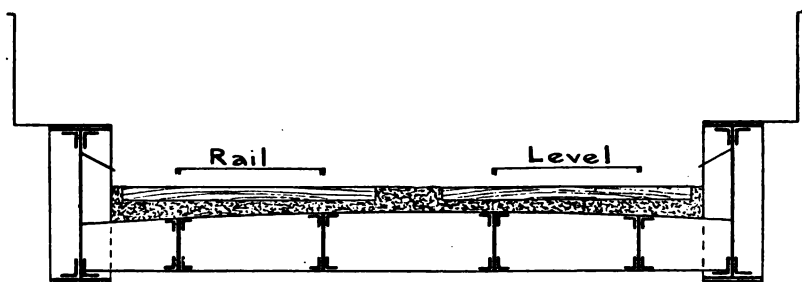


FIG. 4.

adopted in long spans, where the 6 ft. cannot be widened to allow for the centre girder, which is omitted, the cross-girders being made to span the whole width of the bridge. Longitudinal rail-bearers are situated under each rail and a plate floor adopted. Each main girder must be strong enough to carry one line of railway, and theoretically they would be of the same aggregate weight as in a "three-girder" bridge, but the cross-girders would, of course, be heavier. This type of bridge is seldom used, as its erection or

renewal would involve interference with both lines of way at the same time, an operation which prohibits its adoption in situations where there is a large volume of traffic.

The type of underbridge shown in Fig. 5 is an alternative construction to the previous one, and commonly known as a "half-deck" bridge, although it is really a "through" bridge. This design is sometimes adopted in long

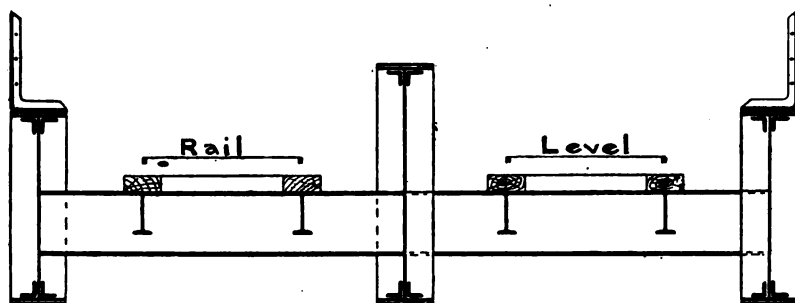


FIG. 5.

spans where there is plenty of construction depth, which allows the floor system to be raised so as to avoid the deep main girders encroaching on the fixed structure gauge. Stools are provided under the ends of the cross-girders, and the latter are also secured to the main girder webs by angle cleats. The rails may be carried over the bridge, either on way-beams bolted to the steelwork, or by the ordinary cross-sleeper ballasted road. It will be seen that if the cross-girders rested on the bottom flanges of the main girders

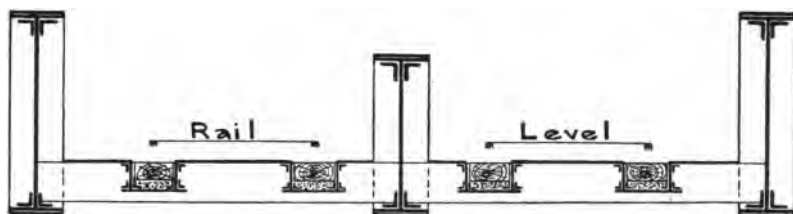


FIG. 6.

the structure gauge would be fouled. To overcome this the main girders would have to be spaced further apart and the 6 ft. space widened out, or the depth made shallow in proportion to the length, which would not be economical.

Tied Floors.—Fig. 6 shows a "through" bridge with a tied floor, the rails being carried over the bridge on longitudinal timbers or way-beams. It is similar in construction to Fig. 1, excepting that the rail-bearers take the

form of troughs, fixed between the cross-girders, in which the sleepers are sunk; the latter being cut into lengths to suit the distance between the cross-girders.

Another type of tied floor is shown in Fig. 7. The rails are carried on longitudinal way-beams sunk into built-up trough girders, which in elevation

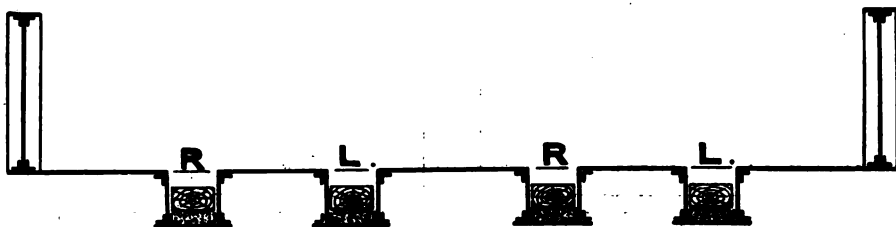


FIG. 7.

are usually hog-backed in shape. The way-beams are sometimes bedded on a layer of sand or cement, to reduce vibration. This type of bridge was at one time largely adopted for spans up to about 25 ft., but beyond this span no advantage can be gained by its adoption, as the depth of the trough-girder becomes too deep for the depth of construction. Both in this type and the one shown in Fig. 6, the disadvantage of a fixed road is apparent.

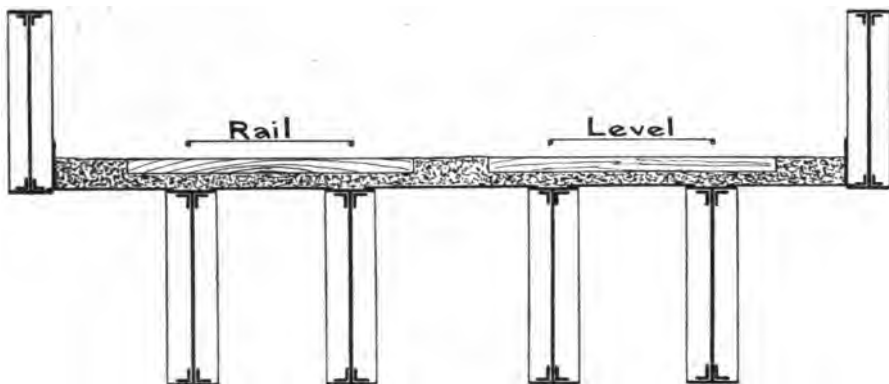


FIG. 8.

Such constructions are very seldom adopted nowadays, and are only resorted to in unavoidable cases of limited construction depth.

Deck Bridges.—A deck bridge is shown in Fig. 8, which can only be used where there is ample construction depth. In this design the permanent way rests on the top flanges of the main girders. There may either be four girders, one under each rail, or three, as shown in Fig. 9. The flooring is

formed of thin plates riveted to the top flanges of the main girders, and the loads are transmitted directly to the main girders without the interposition of cross-girders or rail-bearers. It forms an ideal construction, both as regards first cost and future maintenance, all parts being open to inspection and painting; and riveting at site is reduced to a minimum. In bridges of this type, jack arches of brickwork or concrete are sometimes used, as shown in Fig. 9. They form a very massive, solid, and permanent construction, but are only used in bridges of small or moderate spans, on account of their great weight and cost. However, for bridges up to say 60 ft. span, where this type of construction can be adopted, the extra weight of steel work in the main girders and the cost of the jack arching is more than balanced by the absence of cross-girder and rail-bearer details. Further-

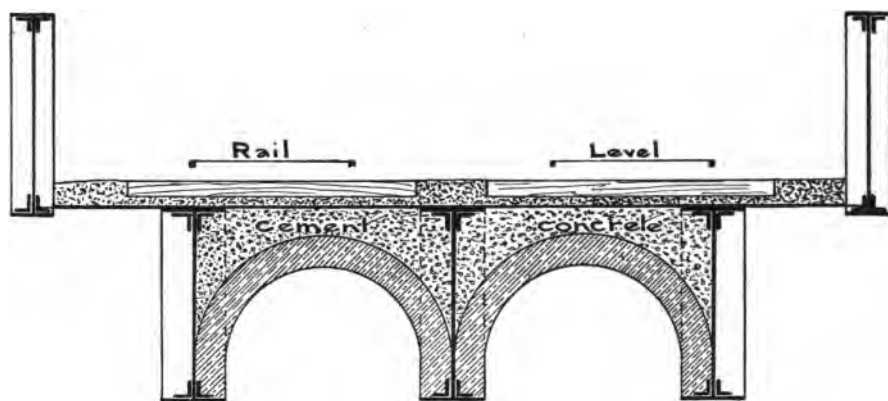


FIG. 9.

more, a great area of the steelwork is protected by the arching; as a result a less area requires painting, and maintenance charges are thereby reduced.

If brick or concrete arches are adopted, the main girders should be designed with a liberal depth, so as to reduce the deflection and be stiff under the load, and the arches should also be built with great care and good materials, to prevent any separation of the brickwork or concrete from the steelwork, and the possibility of water or moisture percolating through and coming into contact with the steelwork; but no danger need be feared in this respect, even under very heavy loads, if the above conditions are fulfilled.

Steel Trough Floors for Railway Underbridges.—Many kinds of steel troughing are used for railway bridge floors. There are various sections,

mostly pressed, but some are rolled, and a number of special types have been patented. Troughing is usually laid transversely to the main girders resting either on the top or bottom flanges, as in Figs. 10 and 11, and sometimes on a ledge or shelf formed by an angle riveted to the web of the main girders, as

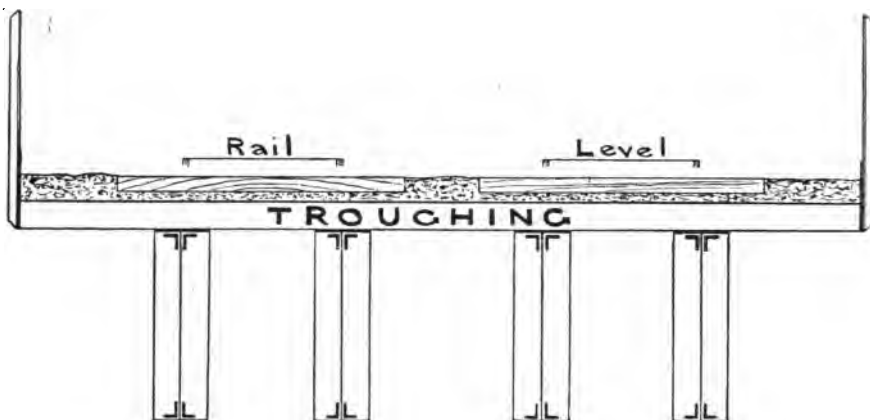


FIG. 10.

in Fig. 12. It may also be used longitudinally, parallel to the main girders, and supported by cross-girders, as in Fig. 13.

Makers of troughing claim many advantages for its use in railway under-bridges. It is relatively light, easily handled, and very rigid; the load is

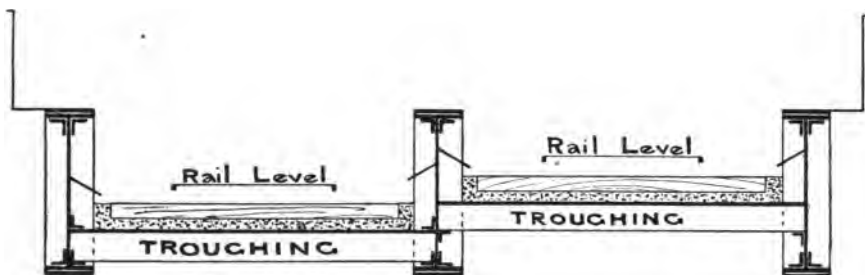


FIG. 11.

FIG. 12.

more uniformly distributed along the main girders, and a continuous surface of uniform strength is presented against possible derailment.

The principal objection to the use of troughing is the great difficulty of arranging a good connection at the ends with the main girders. The best connection that can be made is very inefficient, and it is a usual occurrence to find designs where only very feeble attempts have been made to make any

connection at all, the result being that the load is transmitted in a very objectionable manner. Another objection to its use is the difficulty experienced in providing efficient drainage, as the imperfect connection at the ends allows dirty water to escape and find its way to the steelwork of the main girders, with serious effects. A further disadvantage of troughing

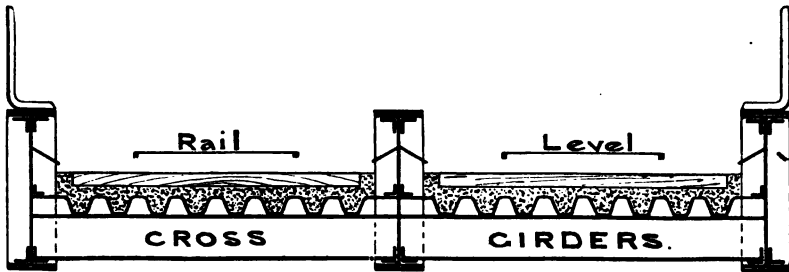


FIG. 13.

is the large amount of site riveting necessary for the connection to the main girders, and joining the sections together.

Troughing is made in various sizes, and the handbooks issued by manufacturers of troughing, etc., usually give the various sizes together with properties and section moduli. Troughing is usually made, so that the pitch of the troughs, *i.e.* distance from centre of top table of one trough to centre of top table of the next, will be some multiple of 4 ins., which is taken as the most usual pitch of rivets. It is often despatched from the makers' works riveted together in sections of three or four troughs, depending upon their size. Longitudinal sleepers or way-beams, forming a "tied" floor, or a

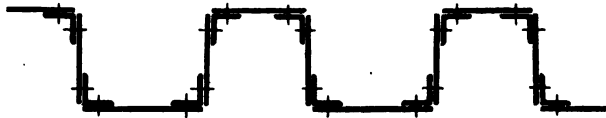


FIG. 14.

"free" cross-sleeper road, may be used, and the troughing covered with ballast or not.

Troughing is very rarely used transversely for the floors of double line bridges, having only two main girders, as the depth of troughing required for the span would necessitate deep, heavy, and very uneconomical sections. Figs. 14 to 17 show the several kinds of troughing commonly used, although there are various forms of patent arched or trough flooring, which are

sometimes adopted. The sections of troughing shown in Figs. 15 and 16 are the forms most generally adopted. There is a certain amount of prejudice



FIG. 15.

against the use of the section shown in Fig. 16, for railway underbridges, as great shearing stresses are apt to occur on the longitudinal seams of rivets joining the upper and lower portions of the troughs. Square troughing,

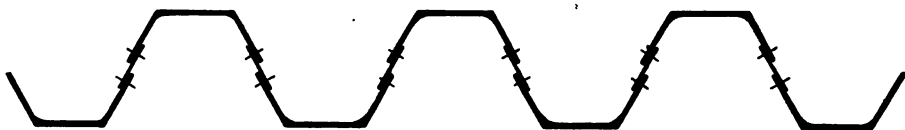


FIG. 16.

built up of plates and sections (Fig. 14) is now seldom used. It is a very uneconomical section, and has been superseded by the more simple and cheaper forms described above. It is often claimed that trough flooring is



FIG. 17.

well adapted for the floors of railway underbridges in cases where there is a very limited construction depth, but this advantage depends upon the manner in which the troughs are used. If, to save construction depth, the sleepers

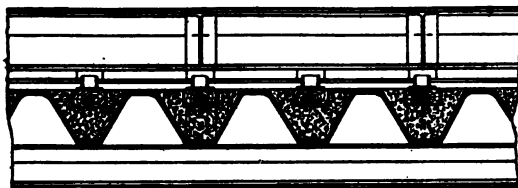


FIG. 18.

are laid in the bottoms of the troughs with the rails just clearing their upper surfaces as shown in Fig. 18, then this practice cannot be too severely condemned. It is, perhaps, the most objectionable of all floor constructions, being noisy, difficult to drain, and extremely dirty; water

collecting in the troughs mixes with the ballast, and as neither can be effectually removed, the pounding of the load soon reduces the ballast to a slurry. The load is also badly distributed, and when the sleepers need renewal it involves lifting the rails to remove them from the troughs. Personal opinion differs as to the advantage or otherwise of troughing for railway bridge floors. Some railway-engineers adopt it to a large extent, while others do not use it at all, except on very rare occasions.

SMALL SPAN RAILWAY UNDERBRIDGES

As in large spans, the same difficulty respecting construction depth is often experienced in the design of small span bridges over occupation roads,

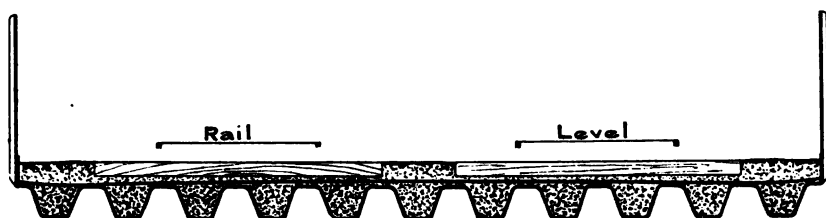


FIG. 19.

cattle creeps, subways, etc. In such bridges pressed steel troughing laid longitudinally, as shown in Fig. 19, is largely adopted, there being no objection to its use in this manner. Of course, the longer the span the deeper the troughs, requiring increased construction depth. The troughing should be

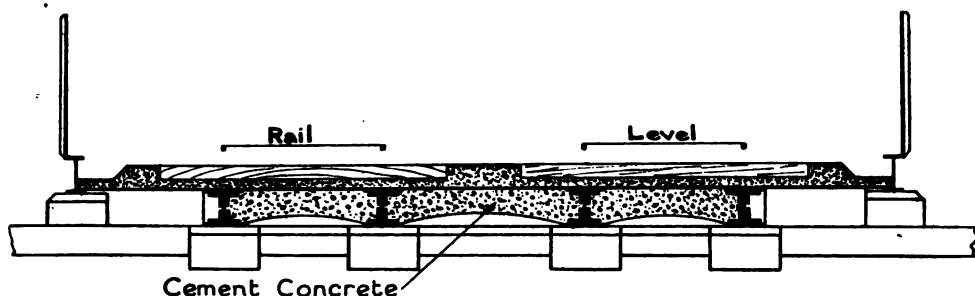


FIG. 20.

stiffened by attaching continuous bearing-plates at the ends on the abutments, and, if necessary, by a continuous tie bar at the centre of the span. This type of bridge forms an economical construction, allowing a cross-sleeper road to be used, but its adoption is limited to small spans.

Fig. 20 also shows a type of bridge used for small spans, with a shallow

girder under each rail, and floor plates attached to the bottom flanges, the spaces between the girders being filled with cement concrete to the level of the top flanges. The outer girders carry very little load, and may consist of rolled steel joists or light built-up girders, the spaces between them and the main girder being covered by floor-plates riveted to the top flange of the girders.

An alternative design is shown in Fig. 21. It is similar to the previous

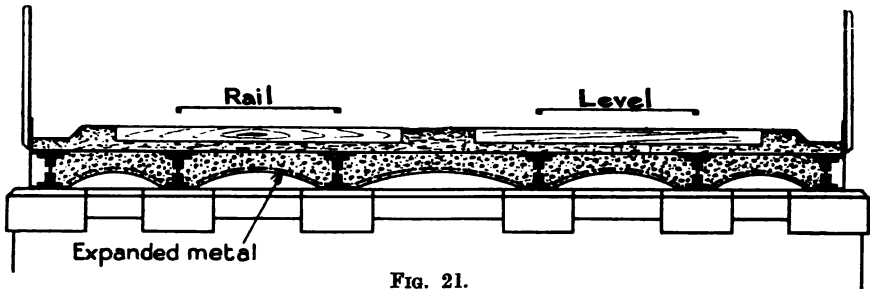


FIG. 21.

one, except that instead of steel floor plates the cement concrete arches are reinforced by expanded metal. This construction has the advantage of reducing painting to a minimum.

When the construction depth is so shallow and limited as to preclude the use of troughing or any other type of girder, allowing a cross-sleeper road to be used, then built-up trough girders must be resorted to, as shown in Fig. 22.

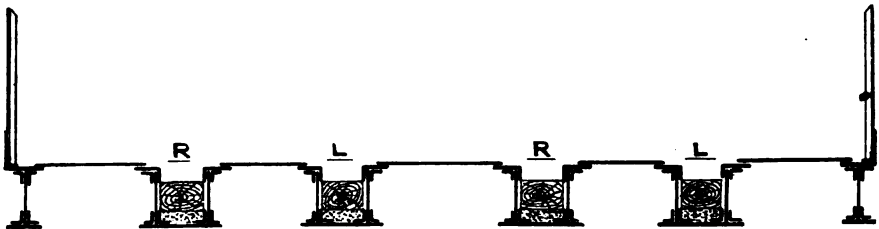


FIG. 22.

An angle fish-plate road is adopted, the rails being carried on longitudinal sleepers resting inside the trough.

Steel floor plates attached to the top flanges of the girders are usually employed, but sometimes this construction is restricted to the spaces between each of the two running rails, the 6 ft. spaces and the outer spaces being covered with timber planking resting upon the bottom flanges. By adopting the latter practice, the riveting at site is reduced to a minimum, as the troughs

and flooring for each road can be delivered to the site in one piece, but it will be found that the extra first cost of providing an all-steel floor will be justified by the absence of renewal of timbers and the maintenance thereof. The disadvantages of this construction are the awkward cutting and packing of the longitudinal sleepers necessary in order to suit the camber of the girders and the cant or super-elevation of the rails, and generally the use of

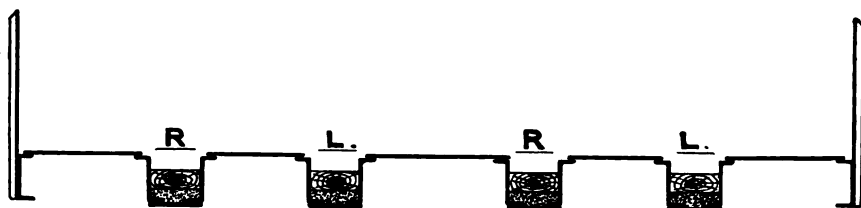


FIG. 23.

special fish-plates to support the rails, necessitating a break in the standard uniformity of the road. Of course ordinary chairs can be used in bridges of the way-beam type, in which cases the cant of the rails is provided for.

Another bridge of the way-beam type is shown in Fig. 23. This is chiefly composed of rolled steel sections, and for spans up to 12 ft. is one of the lightest and cheapest constructions that can be adopted.

TYPES OF ROAD BRIDGES AND FLOOR SYSTEMS

Road bridges and their floor systems are very similar to several of the types of railway underbridges which have just been described.

The types of road bridge floors most generally adopted are as follows :—

1. **Cross Girders and Jack Arches.**—This type of floor, as shown in

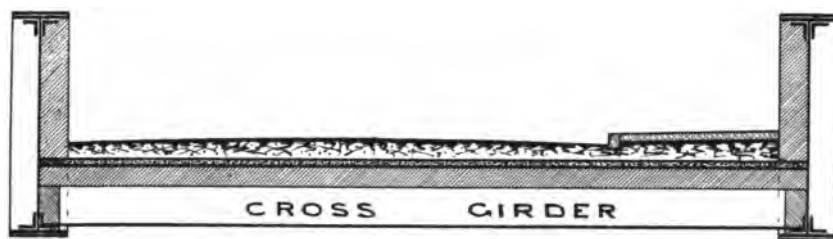


FIG. 24.

Fig. 24, is usually adopted for long spans of medium width, and consists of cross-girders spaced from 4 to 6 ft. apart, carried by main girders. Jack arches of brick or concrete are turned between the cross-girders, and the spandrels of the arches filled in with cement concrete which is carried up

level with the top flanges of the cross-girders. The main girders, if the depth permits, also serve as parapets, if not, a light steel parapet must be provided.

2. Longitudinal Girders and Jack Arches.—When the span is too wide to allow of the adoption of cross-girders, and when there is sufficient construction depth available, the type of floor shown in Fig. 25 is adopted.

It consists of longitudinal girders spanning the whole length of the bridge,



FIG. 25.

and spaced from 3 to 6 ft. centres, according to the depth of the girders. Brick or concrete jack arches are turned between the girders, and brick or light steel parapets provided on the outside girders. The whole of the girders should be well tied together, so as to avoid any movement due to the thrust of the arches.

This is a very usual type of floor for road bridges, and forms an economical, massive, and permanent construction. Little or no site riveting is required,



FIG. 26.

and a great area of the steelwork is protected by the arching, therefore the painting required is very small.

3. Transverse Troughing.—This is a very common type of floor for road bridges of narrow width and carrying light loads. The troughing is carried between the main girder and fixed in the same way as in railway bridges, the main girders acting as part parapets (see Fig. 26). This type of floor is limited to narrow bridges and light loads, as wide bridges would necessitate the use of deep, heavy and very uneconomical sections of troughing.

4. **Cross Girders and Longitudinal Troughing.**—In this type of floor, troughing is laid longitudinally on the top of cross-girders, spaced from

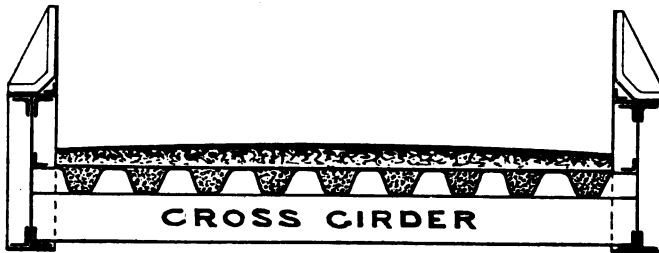


FIG. 27.

5 to 8 ft. apart between main girders (see Fig. 27). This type of bridge is seldom used.

5. **Longitudinal Troughing.**—This type of floor is only used for short spans. The troughing is simply laid longitudinally across the span, no main girders being required. The outside troughs are generally provided with

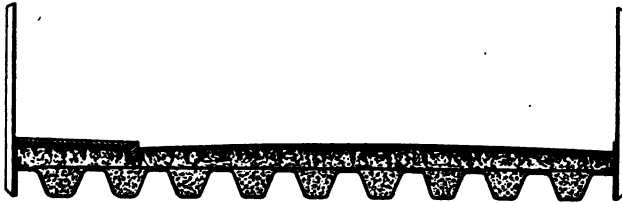


FIG. 28.

open parapets, composed of angle or tee standards and gas tube handrailing (see Fig. 28). It forms a very light, cheap construction, but can only be adopted for short spans. The troughing should be stiffened by attaching continuous bearing-plates at the abutments, and, if necessary, by a continuous tie bar at the centre of the span.

CHAPTER III

THE THEORY OF DESIGN

BEFORE the various portions of a plate girder can be intelligently designed, some knowledge of the theory of beams and girders must be obtained, and the purpose of the present chapter is to indicate this, so far as is required in the design of plate girders.

Case of a Simple Beam.—Considering the case of a rectangular beam, supported at each end, formed of thin layers or strips of material bound vertically, as shown in Fig. 29, a load applied to the beam causes bending, and a consequent tendency to sliding of the layers horizontally.

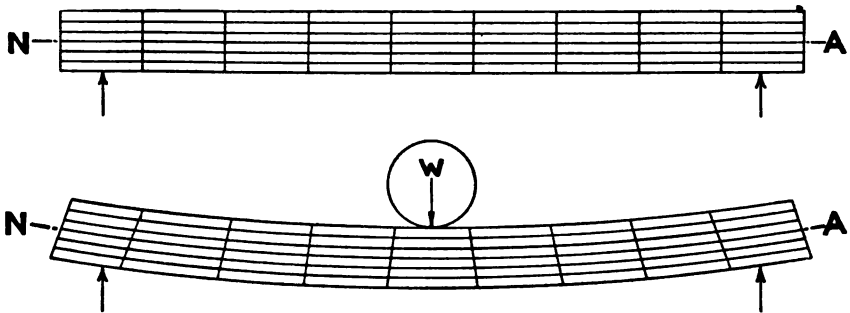


FIG. 29.

The top layers will be shortened or compressed, and the bottom layers lengthened or stretched, the middle layer remaining the same length as before bending; further, it will be found that the amount of stretching and compression will be proportional to the distance of the layer from the middle or neutral layer of the beam.

In text-books on the Strength of Materials it is shown from experiment that the amount of extension or compression produced in a material is, within ordinary limits, proportional to the load producing the extension or compression. Therefore, in the composite beam being considered, the forces in a certain layer, across any section, are proportional to the distance of the layer from the neutral layer; or more precisely, from the neutral axis of the

beam as shown in Fig. 30. The area of each strip being supposed to be the same, the stresses in the strips will be proportional to the forces.

The force or stress is a tensile one below the neutral axis, and a compressive one above, in the case of a simply supported beam, and the intensity of the stress varies from zero at the neutral axis to a maximum at the outer layers. The maximum intensity of tensile stress occurs at the extreme

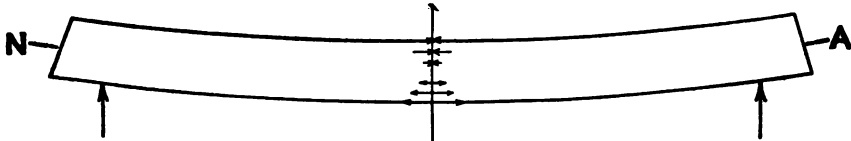


FIG. 30.

bottom layer, and is just equal to the maximum compressive stress at the extreme top layer, as represented graphically in Fig. 31, the shaded areas ABO and CDO, representing the total tension and compression.

As the amount of material in each layer is the same, there is a waste of material in the rectangular section, seeing that the intensity of stress is gradually decreasing from the outer layers to the neutral axis.

The correct section would be as shown in Fig. 32, for here the amount of

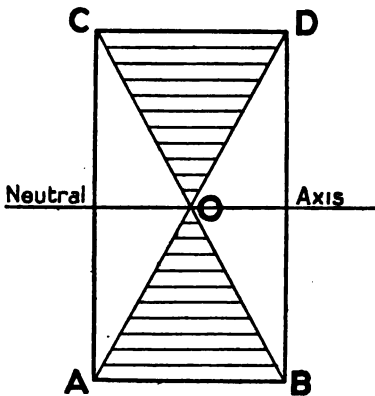


FIG. 31.

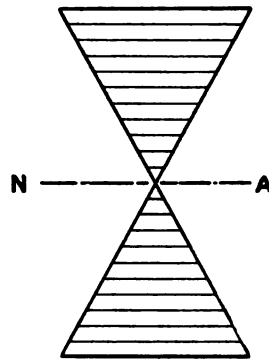


FIG. 32.

material decreases in the same ratio as the stress, thus giving the same intensity of stress per unit area of the material of each layer. All the material of such a section would thus be used to the fullest extent, but the shape of the section is obviously not one which could be used in practice.

However, in practice, this principle is to a certain extent carried into effect, and iron and steel girders are therefore built up of plates and angles as shown in Fig. 33. By this arrangement, as will be seen, the material is

concentrated at the top and bottom flanges where the greatest direct stresses occur.

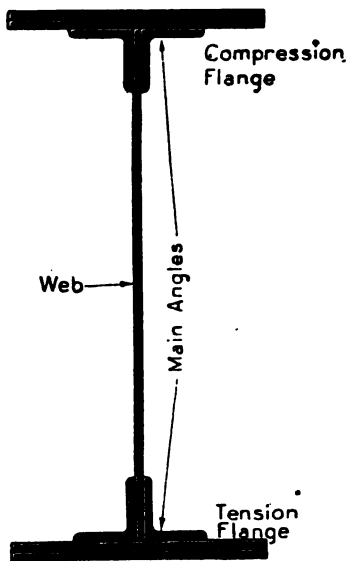


FIG. 33.

The top and bottom flanges consist of one or more plates, depending on the stress to be borne, the top flange taking the compressive stresses, and the bottom flange taking the tensile stresses. The flanges are joined together by a thin web plate, which serves to resist the shearing stress, which will be considered later on.

The following important assumption is made in designing the flanges of plate girders:—

All the horizontal compressive and tensile forces are assumed to be uniformly distributed over the areas of the top and bottom flanges respectively. The area of the web is neglected by most designers, although, of course, horizontal forces exist across the web at this section, just as they do across the flanges.

These distributed forces on the areas of the flanges can be replaced by

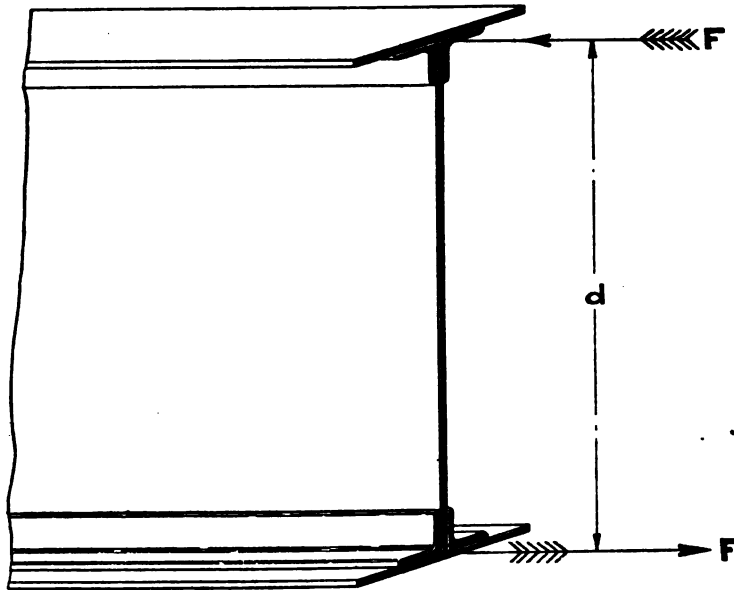


FIG. 34.

their resultant F , Fig. 34, acting at the centroid or centre of gravity of the

flange section, this being commonly taken as being at the backs of the angles connecting the web to the flanges. This approximation is quite accurate enough for practical purposes.

The moment of each of these forces is $F \times d$, where d is the distance between their lines of action, and this moment (called the *internal* moment, because it is produced inside the beam) must be just equal in practice to the

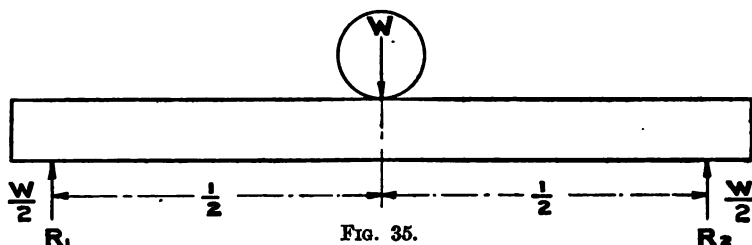


FIG. 35.

external moment, or bending moment at the section being considered, produced by the external loading (Fig. 35).

Suppose the right-hand part of the beam fixed, and consider the effect of the reaction of the support R_1 , on the left-hand portion. This reaction is equal to $\frac{W}{2}$ and acts upwards, tending to bend the left-hand portion of the beam, as shown in Fig. 36, and this bending tendency must be counteracted by the forces F of Fig. 34. The bending tendency of Fig. 36 is the *Bending*

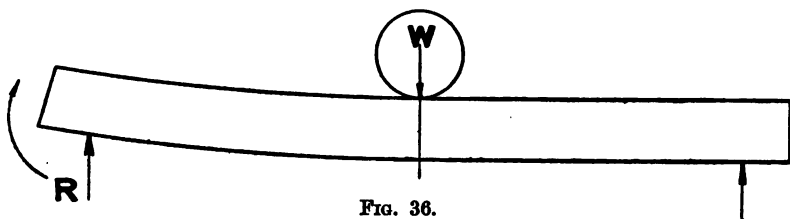


FIG. 36.

Moment produced by the load, and this must be equal to the *Internal Moment* or *Resisting Moment* of the beam. This is represented by the equation :—

$$\text{Bending Moment} = F (\text{total flange load}) \times d (\text{moment arm of flange load})$$

and this holds true for the Bending Moment at any section.

The total flange load F is obtained by selecting a suitable working stress in tension or compression (f_t and f_c tons per square inch), and multiplying by the number of square inches of metal in the cross-sectional area of flange. The following equation represents this relation :—

$$F (\text{tons}) = f_t \text{ or } f_c (\text{tons per square inch}) \times A (\text{flange area in square inches})$$

It follows from the above, that the Bending Moment at any point of a girder divided by the corresponding depth between centroids of flanges, d , gives the total flange load or stress, F , and this divided by the working stress per square inch, gives the net area of metal, A , required in one flange. Care

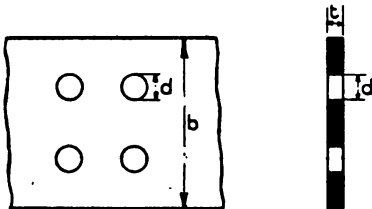


FIG. 37.—Net area of flange plate
 $= (b - 2d)t$.

able to resist the tensile stresses is obviously the net area of the metal, i.e. the area of the plates and angles comprising the flange minus the area of the rivet holes at the section (see Fig. 37). When there are 4 rivets almost on the same centre line (Fig. 38), there is a tendency for the plate to fail, not at a perpendicular section $a a$, but along the line $b b$. To subtract the area of 2 rivet holes, all that actually occur at the section $a a$, or any similar section, would not be allowing for possible failure along the zigzag line $b b$.

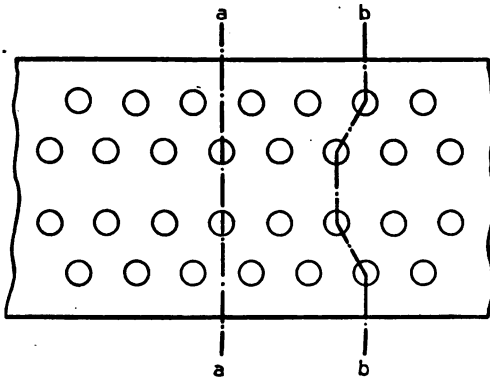


FIG. 38.

To allow for this tendency a compromise is made and the areas of 3 rivets subtracted. All rivet holes should be taken as $\frac{1}{16}$ " larger diameter than that of the rivet. The compression on the top flange is usually regarded as being unimpaired by the rivet holes, and in this case the gross area of the plates and angles comprising the flange is taken as being available in resisting the compressive stresses.

The total flange area, A , is made up of the net (in tension) or gross (in compression) areas of all the flange plates, together with the net or gross area of the pair of main angles which are considered as part of the flange area.

So far as the area of the main angles is concerned, it is becoming the practice to include in this area only the net (in tension) or gross (in

compression) area of the horizontal leg of the angles, as in Fig. 39, in which the net flange area is shown black.

The area of the vertical leg is small, as a rule, compared with the total flange area, and the area is not in the best position to resist bending of the beam and so its effect can be neglected. Further, if the area of these legs is included, the area of the portion of the web between them cannot logically be excluded, which area has a proportional resisting moment to the area of the angle legs. However, the assumption is made that the whole area of the web carries shearing force only, hence it is better not to include it in the flange section, and, therefore, also not to include the areas of the vertical legs of the main angles.

Some designers, however, include both the areas of the vertical legs and

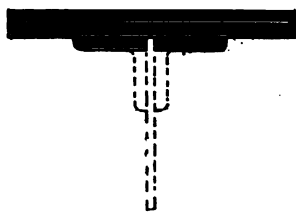


FIG. 39.



FIG. 40.

that of the web between them, as in Fig. 40; the resulting design being certainly more economical in material, but it has a proportionately smaller margin of safety.

In a plate girder the total flange stress is greatest at or near the centre, diminishing towards the abutments, consequently the section of the flanges can correspondingly diminish. This is generally accomplished by varying the thickness of the flanges, the width of the plates composing them being kept the same throughout, but the lengths are so regulated that the sectional area is proportional to the bending moment.

For instance, the bending moments of a uniformly loaded girder can be represented graphically by the ordinates of a parabola. Thus in Fig. 41, the curve AKB represents to any given scale the bending moments at

every point of a uniformly loaded girder of span AB. To find the position where the flange plates may be stopped, it is only necessary to set up successively from the base line AB, to the same scale as the bending moment diagram, the moments of resistance of the main angles and individual flange

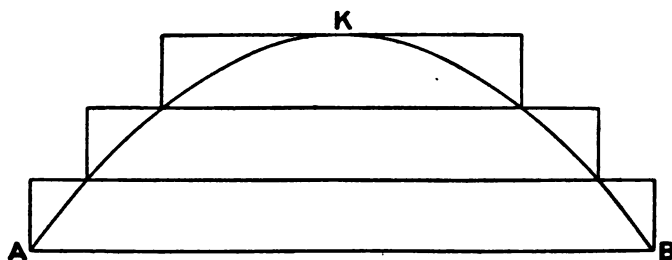


FIG. 41.

plates, as shown. The points where the horizontals cut the bending moment curve give the theoretical lengths of the flange plates, but in practice the plates are extended beyond these points as explained in Chapter V.

ECONOMIC DEPTHS

Referring to the equation $BM = F \times d$, it will be seen that as the bending moment is fixed for any given section with any fixed loading, the value of F , the total flange load, depends on the value adopted for d , the depth between the centroids of the girder flanges. The smaller the value of d the greater will be that of F , and consequently the greater will be the weight of metal in the flanges.

It has been found, as a result of calculation and observations in practice, that the greatest economy of metal is obtained where d is made from one-eighth to one-fifteenth of the span, one-tenth or one-twelfth being a good average value.

The depth of a girder is often limited by considerations of headroom, clearance, etc., but whenever possible the limits given above should not be exceeded. The standard depth should be one-tenth to one-twelfth of the span, and this should not be departed from in a single web plate girder of varying section without due consideration.

If a depth greater than one-eighth span be adopted it will be found that the amount of metal in the flanges will be comparatively small, but that in the web will be very much increased. The reason for this latter is, that although a thinner web, thinner than that required for one-tenth span, would suffice to carry the shearing forces, a thicker one would be required in practice to

allow for the greater buckling tendency due to the increased depth. If a depth less than one-fifteenth is taken, difficulty will be experienced in providing for the heavy end shear on the web.

As explained elsewhere, girders of varying depths are not advisable in ordinary bridge work; they may certainly be of more scientific design, but the extra cost of manufacture nearly always nullifies the saving on the cost of the material. When, however, these girders are used, the depth at the middle can be taken at about one-eighth span, and at the ends about one-twelfth span.

SHEARING FORCE

No consideration has been given up to the present of the effect of the sliding tendency, at every vertical section, which the load W (Fig. 35) produces.

The direction of the force W is downwards, and that of the support reaction is $\frac{W}{2}$ upwards; there is, therefore, at every vertical section between W and the support, a tendency for the left-hand half of the beam to slide upwards relatively to the right-hand half. This vertical sliding tendency is called the Shearing Force at the section, and it has to be balanced by an equal resisting force in the material of the beam.

It is shown in books on Mechanics that a vertical shearing force is always accompanied by a horizontal shearing force of the same intensity. The effect of this horizontal shearing force is a tendency to slide the web relatively to the main angles, and it has to be resisted by the material of the rivets connecting these parts of the girder. The shearing forces, both horizontal and vertical, will be greatest just over the abutment, and it is at this point that the proportions of the web plate and the pitch of the riveting in the angles is determined.

As the shearing forces become less as the section approaches the centre of the girder, the thickness of the web, and also the pitch of the angle rivets, are sometimes changed in proportion; but as practical considerations limit such changes to a great extent, they will be considered later.

Design of Web Section.—In designing the web, a vertical section of the girder over the abutment is taken for the purposes of calculation, because the maximum shearing force occurs at this section. However, the section of the web as calculated to resist this shearing force alone, is nearly always too small, and the criterion of web design is, in most cases, the bearing resistance of the rivets connecting the web to the main angles. Then, as

mentioned in the chapter on Practical Design, purely practical considerations often greatly affect and modify the calculation of the scantling.

Design of Web to Resist Vertical Shearing Force.—

If S = total maximum shearing force in tons.

f_s = safe shearing stress of material in tons per square inch.

d = effective depth of web at the section, in inches.

(i.e. the depth minus the total lengths of the diameters of the rivet holes occurring at or near the section.) (See Fig. 42.)

t = thickness of web in inches.

$$S = d \times t \times f_s$$

$$\text{or } t = \frac{S}{d \times f_s} \text{ (inches)}$$

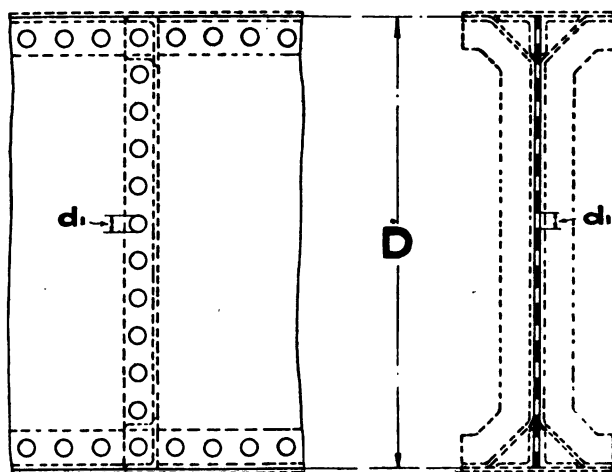


FIG. 42.—Effective depth of section $d = (D - 12d_1)$.

If the line of rivet holes is comparatively near to the section considered, their diameters are subtracted as shown above, and (d) is the net area of metal at the section.

As a stiffener nearly always occurs at the abutments, these holes are always subtracted in considering the resistance of the web to shear. For preliminary calculations the effective depth (d) may be taken as equal to $\frac{3}{4}$ of the total depth.

How the Riveting Modifies the Web Section.—It has been mentioned in the theory of design, and it can be found in text-books on the Strength of Materials, that a vertical shearing force is always accompanied by

a horizontal shearing force of the same intensity. There will, therefore, be a horizontal shearing force of S tons or $\frac{S}{d}$ tons per inch run, on the web at the section considered, and this tends to shear the web from between the main angles, which tendency has to be resisted by the strength of the connecting rivets.

The usual method of calculation is to find the horizontal shearing force per foot run, which in this case is $\frac{12 \times S}{d}$ tons. This proportion of the total load must, therefore, be taken by the rivets connecting the web and main angles in the first foot of length of the girder to the left of the section. In this way the number of rivets required per foot can be calculated as follows:—

If f = value of one rivet in double shear or in bearing, tons.

d = depth of web plate in inches.

n = number of rivets per foot run.

Then
$$f \times n = \frac{12 \times S}{d} \quad \text{or} \quad n = \frac{12 \times S}{d \times f}$$

If n works out to not exceeding 3, 3 rivets per foot, or a pitch of 4" will be required, while if n exceeds 3, but does not exceed 4, a pitch of 3" will be required. If n should exceed 4, then its value must be decreased to 4 by increasing the value of f , as a 3" pitch is a minimum one. It will nearly always be found that the bearing value of these rivets in the web plate will be less than their double shear value, and so the former has to be used as the value of f in the formula above.

This value can be increased, therefore, either by increasing the dia. of rivet or by increasing the thickness of the web plate; 1" dia. can be considered a maximum diameter for the rivets, so any further increase in the value of f must be made by increasing the web plate thickness.

As the intensity of the shearing force decreases towards the centre of the girder, the pitch of the rivets could be gradually increased from the abutments outwards, but for practical reasons this is rarely done. Some designers, however, increase the rivet pitch in this way to a maximum of 6".

Other Stresses on Rivets.—Besides the horizontal shearing stresses on these connecting rivets, it should be remembered that there are often very severe local vertical shearing stresses on them. In the manufacture of a girder, it is frequent practice, and, indeed, almost necessary, to keep the edge of the web plate about $\frac{1}{8}$ " down from the tops of the main angles as shown in Fig. 43.

This means that if any load is applied directly to the flange, as in the case

of a deck bridge, local shearing stresses will be caused in the connecting rivets before the load can be communicated to the web plate. Stiffeners at the point of loading will, of course, relieve this to some extent, but the point should not be lost sight of, in the consideration of rivet stresses.

Web Sections at other Points of the Girder.—Although the thickness of the web may be reduced towards the centre of the girder in certain cases, as a rule no calculations are made for this. The points of change and the thicknesses are fixed from purely practical considerations, and will be discussed in the chapter on Practical Design.

Stiffening of the Girder.—The girder thus built up of web plate, main angles and flange plates now needs stiffening, for two chief reasons:—

(1) The comparatively thin web plate would obviously buckle if the beam were deflected to any extent, and it may be assumed that the buckling

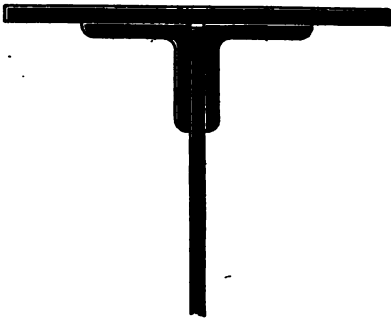


FIG. 43.

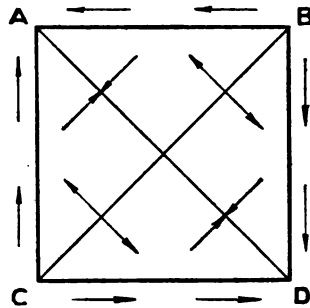


FIG. 44.

tendency is present, though in a less degree, when the deflections are as small as they are in practice.

(2) The flanges require some support to keep them in the correct lateral position, *i.e.* perpendicular to the web plate. Both these effects, the buckling of the web and the twisting of the flanges, are prevented by the use of web stiffeners, which are angle-bars or tee-bars riveted to the web plate, in a vertical position, at various points in its length.

The function of these stiffeners in preventing buckling of the web can be best seen by considering the stresses which are induced in the web plate. It has been shown that these consist of two main stresses—a horizontal shearing force and a vertical shearing force of the same intensity—and in text-books on Mechanics it is shown that these produce two stresses, one tensile and one compressive of the same and equal intensity across planes inclined at 45° to the direction of the original shear stresses, Fig. 44.

If a small square panel of web plate be taken, ABCD, *e.g.* between W and

R₁ Fig. 35, the horizontal and vertical shearing stresses are shown by the lines BA and CD, and CA and BD respectively. It will be seen that the result of these forces acting together is to produce a tensile stress across diagonal CB and a compressive stress across diagonal AD for the particular panel chosen. The intensities of these stresses will be just equal to the intensities of the original shearing stresses.

The web tends, therefore, to buckle in the direction of the compressive stress, i.e. across diagonal AD, at 45° to the horizontal. It can, therefore, be assumed to be composed of a number of struts inclined in this direction, as shown in Fig. 45.

Numerous theories have been propounded for calculating the spacing of stiffeners, all of them being based on this strut assumption. At first sight there seems to be all the data at hand for calculation; the load on the strut is known, also its length, and its thickness, so it is easy to find the minimum width (unsupported), required for safety, if a safe working stress is assumed,

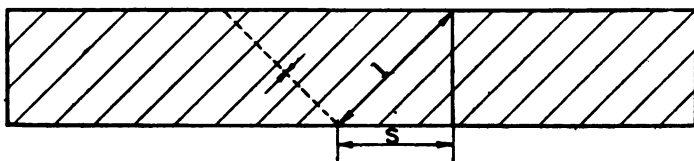


FIG. 45.

or if a width is assumed, the greatest unsupported length allowable can be found.

If l = greatest unsupported length allowable,
and s = spacing of stiffeners (centre to centre) (see Fig. 45).

Then
$$s = \frac{l}{\sqrt{2}}$$

Formulae of the Rankine, or Gordon types, are used in the calculations.

But unfortunately for theories of this kind, practical experience has shown conclusively that they are of little use; girder webs have shown no signs of buckling long after the limit of safe theoretical load has been passed. The reason for this is not far to seek; each strut is considered in the theory, as acting independently, but this is not the case in practice, as the tensile stress between them at right angles offers a great deal of support to the struts and prevents buckling from taking place. As this support is given along the whole length of the strut, it follows that the piece of web supported in this way is far stronger than an independent strip. The practical designer rarely uses theories or formulae in spacing stiffeners, the spacing is so dependent on

other considerations; but he should know, at any rate, exactly what is taking place, and should base his design on the results of theory, even though he gives practical considerations the first place. This point will be referred to later in Chapter VII.

The stiffening of the flange plates against twisting is not generally considered independently, the web stiffeners being used for this purpose, by giving them a connection, in wide flanges, to the outer edges of the plates. In narrow flanges they are just made to bear on the undersides of the flange plates.

Stiffeners are also of use in distributing a heavy, concentrated load over the area of the web plate, and should, therefore, be placed where loads of this character occur, viz. at the edge of the abutment, and the points of cross-girder connections.

CHAPTER IV

LOADS ON BRIDGES AND WORKING STRESSES

THE loads on bridges can be divided into two distinct classes : (1) Dead Loads, (2) Live Loads. As the names imply, the former are the constant loads on the bridge, caused by the weight of the superstructure itself, *i.e.* main girders, cross-girders, rail-bearers, etc., together with rails and their accessories, ballast, or other road material ; whilst the latter are the moving loads caused by the traffic passing over the bridge. In the present chapter it will be explained how these loads are calculated from given data, and how they are affected by various conditions.

DEAD LOADS

The chief dead load on a bridge is the weight of the bridge itself, and, of course, no exact estimate of this weight can be made until the design is completed. In making the preliminary calculations for a bridge an approximate weight of the girders must be obtained, and this weight should be checked when the design is completed, and if any great difference is found between the actual and assumed weights, the design must be amended accordingly. There are several rules by which the weight of a plate girder may be calculated, and which are given in various text-books ; but a practical man can usually estimate the weight of a girder of a given span and type quite accurately enough for the purposes of design, as to a great extent he is guided by his previous experience. It is a good practice for designers to prepare a table giving the span, depth, loads, and weight of as many examples as he can of actual girders of which the weights are known, as such a table would be found exceedingly useful in designing similar girders. The weight of steelwork in a bridge depends on so many factors, such as intensity of live loads, working stress adopted, type of bridge adopted, that it is practically impossible to formulate hard-and-fast rules to take the place of practical judgment and experience.

After an approximate figure has been found for the dead weight of steelwork, the weight of the superimposed material must be added to it, in order to get the total dead load.

The following are average figures used for these latter calculations :—

Weight of ordinary railway ballast	...	...	...	120	lbs.	per cubic ft.
" " concrete	...	...	...	140	"	" "
" " brickwork	...	...	...	120	"	" "
" " masonry	...	...	...	140	"	" "
" " asphalt	...	...	...	156	"	" "
" " timber	...	...	...	45	"	" "
" " granite paving	...	...	...	160	"	" "
" " macadamized road	...	...	...	120	"	" "
" " permanent way for a single line (exclusive of ballast)	...	...	...	175	"	" lineal ft.

Other weights of miscellaneous materials are given in the Appendix.

The total dead load is usually calculated afterwards as a value per lineal foot of the span, and in ordinary cases it will be found to work out at between about $\frac{1}{2}$ ton per lineal foot for spans up to 12 ft., and about 1 ton per lineal foot at about 60 ft. span.

LIVE LOADS

The question of the amount of live load to be allowed for in the design of bridges is a much more complex one than that of dead loads, which has just been considered. This is chiefly because the stresses and strains induced by these loads are much greater than those induced by a load of the same intensity when at rest on the bridge, and also because these stresses are varying ones, that is, they may vary both in magnitude and description as the load passes over.

The amount of variation in the stress due to the passage of a rolling load could be easily determined, provided that the load travelled quite smoothly ; however, in the case of a train running over a bridge, hammering action is produced by uneven permanent way, lurching of the engine, etc., and the extra effect of this can only be estimated.

Also it should be noticed that the load comes on the structure suddenly, and if the span is only a short one, the full live load effect is induced in a fraction of a second.

We will now consider the effect of all these conditions : (1) Varying stresses, (2) Hammering actions, (3) Sudden loading, on the material of the bridge, which, of course, has to be strong enough to resist the stresses produced by them. As regards the effect of varying stresses it has been conclusively proved by the experiments of Wohler, Bauschinger and Stanton (see text-books on Strength of Materials), that a piece of material, acted on by a load varying from zero to a maximum tension, will fail after a certain number of repetitions under a stress of roughly about half that of the same material in direct tension. Then if the stress be made to vary from a tension to compression, the material will fail at a much lower stress still, the value depending on the range of stress.

These experiments and their results should be studied carefully by all engaged in the design of structures, as their significance can hardly be over-estimated.

Then, as regards sudden loading, it can be shown theoretically that if a load is suddenly applied to a piece of material, the strain produced will be just twice that produced by the same load gradually applied, and, therefore, the equivalent stress will be twice that produced by the same load gradually applied, as calculated in the usual way. This effect and also those hammering effects produced by the train, as mentioned above, are usually grouped together under the heading of "Impact" effects, though they are not real impact effects. To produce real impact in a material, it must be acted upon by a load possessing kinetic energy, and which energy must be expended in straining the material. For example, if a weight which is falling is suddenly brought to rest by the tightening of a chain attached to it, the chain will suffer an "impact" effect, as not only is strain produced in it by the hanging weight, but the kinetic energy of the falling weight is also expended in producing further strain in the chain. It is difficult to say how much real "impact" effect there is on a bridge structure; but bridge engineers, as explained above, include under the term "Impact," the effects of sudden loading and the lurching and hammering actions. As it is quite impossible to calculate the extra stresses induced by these secondary effects of the live load, the only alternative is to make some certain allowance for them. There are two methods of doing this, and these are as follows:—

(1) To add a percentage amount (varying with the span) to the actual live load, and to treat the augmented load as a dead one. With this method the ordinary safe static working stress for the material is used, as the added load is assumed to produce the same extra stress as the secondary effects produce in the actual structure.

(2) By reducing the working stress used by means of a formula, which allows for the effect of varying stress or sudden loading. It is not necessary to allow for both varying stress and sudden loading, as the working stress so obtained would be absurdly low. The experiments, mentioned above, on the effects of repeated and varying stresses were really experiments on sudden loading as well, as the fluctuations of stress were fairly rapid. It is therefore assumed that the formula based on the results of these experiments covers both effects.

1st Method.—In using the first method, a difficulty arises as to the exact percentage amount which should be added to allow for the effect of the live load. It should be explained that the static live load is taken to be that of a train of the heaviest locomotives in use, coupled together, and long enough to

completely cover the span. The axle loads and their spacings have to be obtained from the locomotive department.

Plate girder bridges are usually designed from a uniformly distributed load as data, and as the effect of a series of axle loads on a bridge is different from that of a continuous or uniform load, the actual weight of the engines is modified as explained in the next chapter.

When this equivalent load has been found, the amount which is to be added to cover the varying stress and impact effects must next be decided so that the actual working live loads can be determined.

It is explained in text-books on the Strength of Materials that the effect of a load suddenly applied to a material (without real impact) is just double that of the same load applied gradually. Consequently, for rail-bearers and short spans, where the application of the live load is practically instantaneous, the percentage allowance for "impact" is taken as 100 per cent. As explained above, this is the effect of sudden loading, and not real impact, but the terms have been confused and now are practically identical in meaning. The longer the load takes to come on the bridge, the less will be the sudden loading effect, so the percentage can be reduced on longer spans, though opinion differs as to the exact amount of reduction. The following table gives the average values used in modern practice. When this percentage has been added to the uniformly distributed static live load the result is the working live load, and with this method the full safe static working stress can be used in proportioning the sections (see Example III.).

Span. Feet.	Impact allowance. Per cent.	Span. Feet.	Impact allowance. Per cent.
5	100	40	88
8		45	86
10		50	85
12		55	84
15	99	60	83
20	98	65	82
25	97	70	81
30	94	75	80
35	92	80	79
	90		

It might be noticed that the actual effect of impact is greater on the main girders of a deck bridge than those of a "through" bridge, and also that the effect on a ballasted bridge will be less than that on a way-beam floor type, owing to the cushioning and distributing effect of the ballast in the former.

2nd Method.—With this method, that of varying the working stress according to the relative amounts of the dead and live loads, formulæ of the Launhardt-Weyrauch type, as given on page 42, are used.

These formulæ, and indeed this method altogether, are more suitable for use in open web girders than plate girders. In the former the working stress in each member can be proportioned to suit the variation of stress in the member, but in the case of plate girders, the whole girder must be treated as one member and the one working stress found must be used throughout.

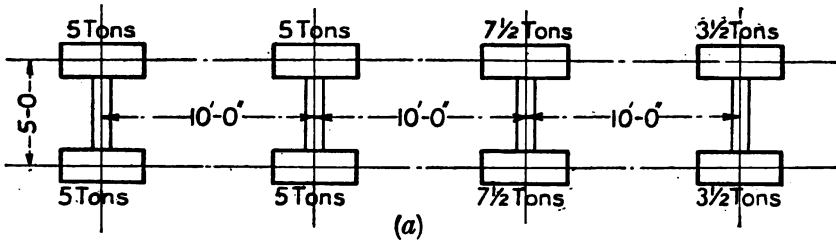
The author would recommend the first method, as being the best one for general use in plate girder design.

Live Loads on Railway Overbridges or Road Bridges

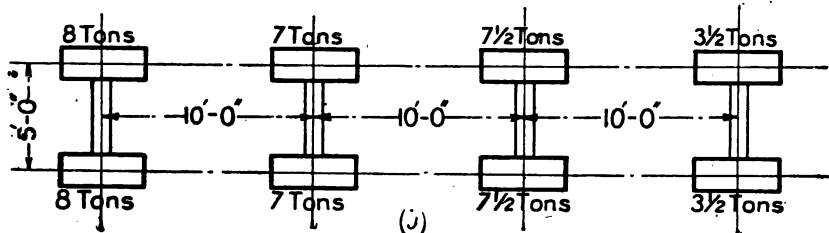
In the case of bridges carrying roads, the effect of impact is not so important, as the maximum live loads on these do not move at great speeds, and their maximum effect is practically no greater than their static loads.

The average values of the static live loads to be used in the calculations for these bridges are given below; they depend on the class of bridge, whether turnpike, public road or occupation road, and also on the local traffic of the district. In manufacturing districts, heavy loads, such as boilers, guns, castings, machinery, etc., are often put on trolleys for transportation by a traction-engine, and these would form the type of loads for the bridges.

Footbridges (country district) ...	...	...	...	100 lbs. per sq. ft. distributed
" (in towns) ...	...	...	...	120 " " "
Occupation Roads ...	...	...	...	140 " " "



Public Roads and Highways (country districts) ... 140 lbs. per sq. ft. distributed and concentrated loads (a).



Highways (in towns and manufacturing districts) ... 168 lbs. per sq. ft. distributed and concentrated loads (b).

The distributed loads given above may be caused by a dense crowd of people walking over the bridge, and the concentrated loads by a traction-engine and wagons passing over at the same time. The results of these loads must be determined separately, as explained in the next chapter, and the results added together to get the combined live load.

As it would be impossible for a traction engine to be crossing a bridge while it was wholly covered by a crowd of people, it is common practice to take the 140 lbs. per sq. ft. distributed over the area of the footpaths only.

Road bridges should be designed to carry their respective live loads in any position upon them, and if their width admits of two such loads passing one another, they must be allowed for.

Local circumstances usually determine the live loads to be adopted in any one case; the heaviest loads passing over the bridge must be allowed for, and also a good margin left for contingencies. The live loads recommended above may be taken as ample for the cases given.

The increasing use of heavy motor lorries for transport must not be overlooked, as although these as a rule do not give such heavy concentrated loads as a traction engine or a steam roller, they run at speeds up to 12 miles an hour, and produce a corresponding impact effect on the bridges.

When a bridge carries an electric tramway the weights on the axles of a car should be ascertained and the rolling load taken as a train of cars coupled together and long enough to completely cover the span.

WIND PRESSURE

In the early days of structural engineering the effect of wind pressure on structures was only imperfectly understood, and was probably allowed for in a merely rough and ready manner. The Tay Bridge disaster, in 1879, brought the subject into greater prominence, and Engineers began to see the need for a more scientific treatment of the stresses caused by wind forces, stresses sometimes greater than those caused by the dead and live loads. During the construction of the Forth Bridge (1883-1890) gauges of various areas were erected at different altitudes and records of wind pressure were taken. These records have also been continued since the completion of the bridge, and form some valuable data for Engineers. The results will be found in an excellent paper by Mr. A. Hunter, M.Inst. C.E.* The greatest pressure ever recorded was 65 lbs. per sq. ft. on a gauge at a height of 387 ft. above sea-level, and the maximum pressure on gauge at 50 ft. height was 20 lbs. per sq. ft.

* Trans. Junior Inst. of Engineers, 1906.

Also it was found that a large area gauge gives a lower figure for the mean wind pressure than the pressure on portions of the gauge, thus showing that high pressures are restricted to comparatively small areas. Other experiments by Dr. Stanton of the National Physical Laboratory, on large models of girders, show that the suction effect of the wind on the lee side of a girder increases the direct pressure on the wind side. On a large model of a lattice girder the total pressure was found to be 1.26 times the pressure (on a rectangular board of equal area).*

The wind load on a structure is evidently a live load, and may produce an impact effect, and the only question to be considered is that of the intensity of pressure to be allowed. It must be remembered that not only is one side of the structure itself open to wind pressure, but also one side of a train which may be passing over. The area on which the wind may be taken to act should thus be the area of the side elevation of the bridge with the train passing over it. The average intensity of wind load should be taken at 30 lbs. per sq. ft., taken on this side elevation of the bridge and train, and the load should be treated as a live load. This will give, for moderate spans, an equivalent dead load of about 56 lbs. per sq. ft., which is the Board of Trade rule for structural design. There is no mention in this rule as to whether this 56 lbs. per sq. ft. is to be treated as a live load or a dead load, but in the light of experiment, a designer is quite justified in using this wind pressure as a dead load.

Of course, there may be circumstances which require higher, or will permit lower, values to be used, and each case must be judged on its merits. Viaducts near the coast and in mountainous countries are often subject to sudden squalls for which it may be prudent to allow a higher wind load. The figure of 30 lbs. per sq. ft. treated as a live load will cover all but very exceptional circumstances, as in the very heaviest gales this figure is but rarely reached, and then such a pressure would be purely local; that is, it would act on a few square feet only. In short spans of less than 40 or 50 ft. the wind pressure can be neglected altogether. The wind load is taken as a uniformly distributed live load acting on the side of the structure, i.e. at right angles to the main live load, and the calculations for the stress produced in the main girders and in the wind bracing can be followed from Example III.

In a "through" bridge the wind bracing can only be placed across the bottom flanges, and the wind load is generally taken as being carried by the bottom flanges of the main girders only. In a deck bridge, where bracing can be put in at the top flanges as well, the wind load can be taken as divided between the two systems of bracing.

* See papers by Dr. Stanton, vols. clvi. and clxxi of the Proc. Inst. C.E.

Working Stresses.—The working stresses adopted in the design of steel bridgework vary to a very great extent with different Engineers, and there appears to be no uniformity whatever as to the values allowed for the various loads and stresses.

The working stress required by the Board of Trade for steel bridges is 6·5 tons per square inch for both tension and compression, which stress must not be exceeded by combining the dead load and heaviest live load stresses.

No mention is made as to whether this figure is intended to cover all impact effects, or whether any allowance should be made for these by increasing the given static load, also no mention is made as to what figures to allow for either shearing or bearing stresses. The whole requirement is left to Engineers to interpret as they think proper, hence the differences in the working stresses adopted.

Assuming the Board of Trade requirement of 6·5 tons per square inch to be the maximum allowed, if this figure were adopted for all spans irrespective of length, then it can be shown that such a course would be irrelevant and incorrect.

In a long girder, say of 250 ft., the range of stress and the effects of impact are both small compared with a short girder, say of 30 ft., where the effects of impact and the range of stress are both high, consequently if a working stress of 6·5 tons were adopted in both cases, the short girder would be unduly stressed.

The average ultimate tensile strength of mild steel required by the Engineering Standards Committee is 30 tons per square inch, so the Board of Trade working stress of 6·5 tons per square inch gives a factor of safety of between 4 and 5.

If an impact allowance is added to the actual live load, and the augmented load treated as a simple static load, as explained on page 37, the ordinary safe static working stress for the material is adopted. This is taken at 7½ tons per square inch for both tension and compression, which gives a factor of safety of 4 with material, having an average ultimate tensile stress of 30 tons per square inch.

When the second method is adopted, that of varying the working stress according to the relative amounts of the dead and live loads, the following formulæ are used :—

(1) *Launhardt-Weyrauch Formula*—

$$\text{Allowable working stress} = F \left(\frac{(2 \times \text{total load}) + \text{dead load}}{(3 \times \text{total load})} \right)$$

where F = safe static working stress
and total load = (live load + dead load).

$$(2) \quad \text{Allowable working stress} = \frac{F}{2 - \frac{\text{dead load}}{\text{live load} + \text{dead load}}}$$

where F is as above. In these two formulæ the safe static working stress is taken at $7\frac{1}{2}$ tons per square inch.

(3) A formula advocated by Prof. Unwin, and extensively employed on some foreign railways—

$$\text{Allowable working stress} = \frac{9 (\text{live load} + \text{dead load})}{2 \text{ live load} + \text{dead load}}$$

In this formula the safe static working stress is taken at 9 tons per square inch.

(4) A formula adopted by one of the leading English railways is as follows :—

When the live load is more than half the total load the allowable stress in tons per square inch

$$= 3 \times \frac{\text{live load} + \text{dead load}}{\text{live load}}$$

This formula gives low unit stresses for cross-girders and rail-bearers, and is equivalent to limiting the range of stress to 3 tons per square inch.

The latter formula (4) is called generally Prof. Cain's formula, but its original form is—

$$f = \frac{\text{safe static working stress}}{2} \times \frac{\text{live load} + \text{dead load}}{\text{live load}}$$

where f = allowable working stress.

In each of the above formulæ the allowable working stress applies to both tension and compression members, all other stresses being reduced in the same proportion.

On some railways no special formulæ are adopted, the allowable working stress varying between certain fixed limits, and the following are the usual values adopted :—

In girders over 40 ft. span—

Tension 6 tons per square inch.

Compression 5 tons per square inch.

In girders under 40 ft. span, including cross-girders and rail-bearers—

Tension 5 tons per square inch.

Compression 4 tons per square inch.

In designing railway underbridges, the last formula, (4), appears to be the one best adapted to cover the extra stresses due to impact, etc.

Shearing Stresses in Webs.—Many formulæ have been deduced for the safe shearing unit stress in web plates, but the one most commonly used is called Cooper's formula, which is as follows :—

$$\text{Allowable shearing stress} = \frac{\text{safe static working stress in shear}}{1 + \frac{d^2}{1500t^2}}$$

where d = distance of stiffeners apart in inches.

t = thickness of web in inches.

The actual stress to be allowed for

$$= \frac{\text{Shear in tons at section under consideration}}{\text{Effective depth of web in inches} \times \text{thickness of web in inches}}$$

The safe static working stress in shear is taken at $5\frac{1}{2}$ tons per square inch.

Many engineers specify that the maximum web stress shall not exceed half the allowable tensile unit stress.

Too much reliance cannot be placed in empirical formulæ for web stress and the spacing of stiffeners, as the former is to a great extent indeterminate, while the latter depends in many cases on practical considerations.

For short span bridges, and in cross-girders and rail-bearers, the maximum web stress should not exceed $3\frac{1}{2}$ tons per square inch, and in no case should the web be less than $\frac{3}{8}$ " thick.

Working Stresses in Rivets.—The safe static working stresses recommended for rivets are—

Shear	...	...	...	$5\frac{1}{2}$ tons per square inch
Bearing	...	...	...	12 „ „

The strength of a rivet in double shear to be taken at twice that of a rivet in single shear. Some authorities consider a rivet in double shear to be only $1\frac{3}{4}$ times the strength of a rivet in single shear, but there is no reason to suppose it is not equal to twice the strength of a rivet in single shear.

CHAPTER V

PRELIMINARY CALCULATIONS IN DESIGN

Introduction.—Now that the loads on bridges have been definitely fixed, a start can be made with the preliminary calculations necessary for the design of the structure.

These consist in the determination of the maximum Bending Moment and the maximum Shearing Force which the given loads produce in the bridge. The definition of these terms, and their determination in simple cases, is outside the scope of the present work, and the reader is referred to any modern text-book on Applied Mechanics.

A knowledge of these terms and their applications will therefore be assumed, in a general way, and the present chapter will only describe the particular methods of determination used in bridge design.

When the total dead load on the bridge has been determined it must be proportioned among the girders before any calculation of B.M. and S.F. is made.

Where there are only two girders, the whole dead load, including the weight of the steelwork itself, can be taken, and divided into halves, one half being carried by each girder.

If, however, there are three girders, two outer and one centre, the two sections of the dead load are best taken separately; the dead load of track, flooring, ballast, etc., is carried in the proportion of one-half on the centre girder, and one-quarter on each of the outer girders, then the estimated weights of the steelwork in each of the main girders can be added afterwards in their proper places.

This is a more accurate method than a simple division, in the proportions of one-half and two-quarters of the total dead load, including all steelwork. Similar methods can be applied to other cases of multiple girders.

DISTRIBUTION OF LOADS

Calculation of Bending Moment and Shearing Force for Dead Loads.—The maximum bending moment (B.M.) on a freely

supported beam or girder covered with a uniformly distributed dead load is—

$$= \frac{WL^2}{8} \text{ (foot-tons) (1)}$$

where W = dead load per ft. of length of span (in tons).

L = length of span in feet.

and this maximum B.M. occurs at the centre of the span. At this point it may be noted that the length of span used in the above formula is the "effective span"; this is approximately equal to the distance between centres of bearings, and can be taken as such for the purpose of calculation. Of course, at this stage the lengths of the bearings will not have been fixed, and an effective span will have to be fixed from previous experience of spans of similar size.

The following table gives approximate values, but these could be modified in special cases :—

Clear span.	Effective span.	Clear span.	Effective span.
10 ft.	11 ft.	50 ft.	52.25 ft.
20 "	21.5 "	60 "	62.5 "
30 "	31.75 "	70 "	72.75 "
40 "	42 "	80 "	83 "

The maximum Shearing Force (S.F.) occurs just immediately above the edge of the abutment, and for a uniformly distributed dead load—

$$\text{Maximum S.F.} = \frac{WL}{2} \text{ (tons) (2)}$$

Uniformly Distributed Live Loads.—In the case of a uniformly distributed live load crossing a bridge the B.M. at any point gradually increases as the load travels over, and it reaches a maximum at the centre when the live load completely covers the span.

Now in the case of *plate girders*, it is only required to know at first the maximum bending moment and the maximum shearing force, and these values are, for a uniformly distributed live load completely covering the span, precisely the same as those produced by a dead load of *the same intensity*.

In order to meet the impact effect of the rolling loads they must be increased as explained in the last chapter, and they can then be regarded as dead loads, and the B.M. and the S.F. produced by them is calculated in the same way, and by the same formulæ, as those used for the dead load. See Formulæ (1) and (2) above.

(If the alternative method of reducing the working stress to allow for

"impact" is used, the live load used here will just be equal to its actual amount, and the allowance made later.)

Case of a Series of Concentrated Loads.—In the case of a series of concentrated loads, such as the axle loads of a train of locomotives passing over a span, the calculations for finding the maximum values of the B.M. and the S.F. are extremely tedious, and graphical methods, which give fairly accurate results, are commonly used. The results are, in any case, as accurate as the conditions warrant.

The general method is to find the *actual* maximum B.M. and S.F. for any given span, and then calculate the value of the uniformly distributed live load which would give the same maxima in each case. The equation for maximum B.M. representing this is—

$$\text{Max. BM} = \frac{W^*L^2}{8}; \text{ or } W_e = \frac{\text{Max. BM} \times 8}{L^2}$$

The value W_e is now the equivalent uniformly distributed live load per ft. run for the span of L feet, which would give the same maximum B.M. as the series of axle loads produce.

This can be done for a series of spans from, say, 5 ft. to 80 ft., and a table of Equivalent Uniformly Distributed Live Loads drawn up from the values found. Once this table is compiled it is always available for future reference, and laborious calculations are thereby saved.

All that is done in designing for a span of a given size is that the Equivalent Uniformly Distributed Live Load is found from the table, and the maximum B.M. and S.F. calculated as in the previous section.

Certain precautions are necessary in using these tables, and they will be referred to later on.

Preparation of Table of E.U.D.L. Loads.—It will now be shown how a table of Equivalent Uniformly Distributed Live Loads is prepared from a given series of axle loads and spacings.

The axle loads and spacings must be obtained from the locomotive department and will represent the heaviest loads which the bridge will have to carry. The "type train," as it is called, consists of a series of these locomotives coupled together. For an example, a span of 40 ft. will be taken, but a number of different spans can be done on the same diagram.

Referring to Fig. 46, the type train of parts of two engines coupled together is set down on a horizontal base line to a suitable scale (say 1" = 8 ft.). *The heaviest axle loads should be set down about the middle of the line.* To the left side of this base line a vertical line is drawn, and the axle loads are set down

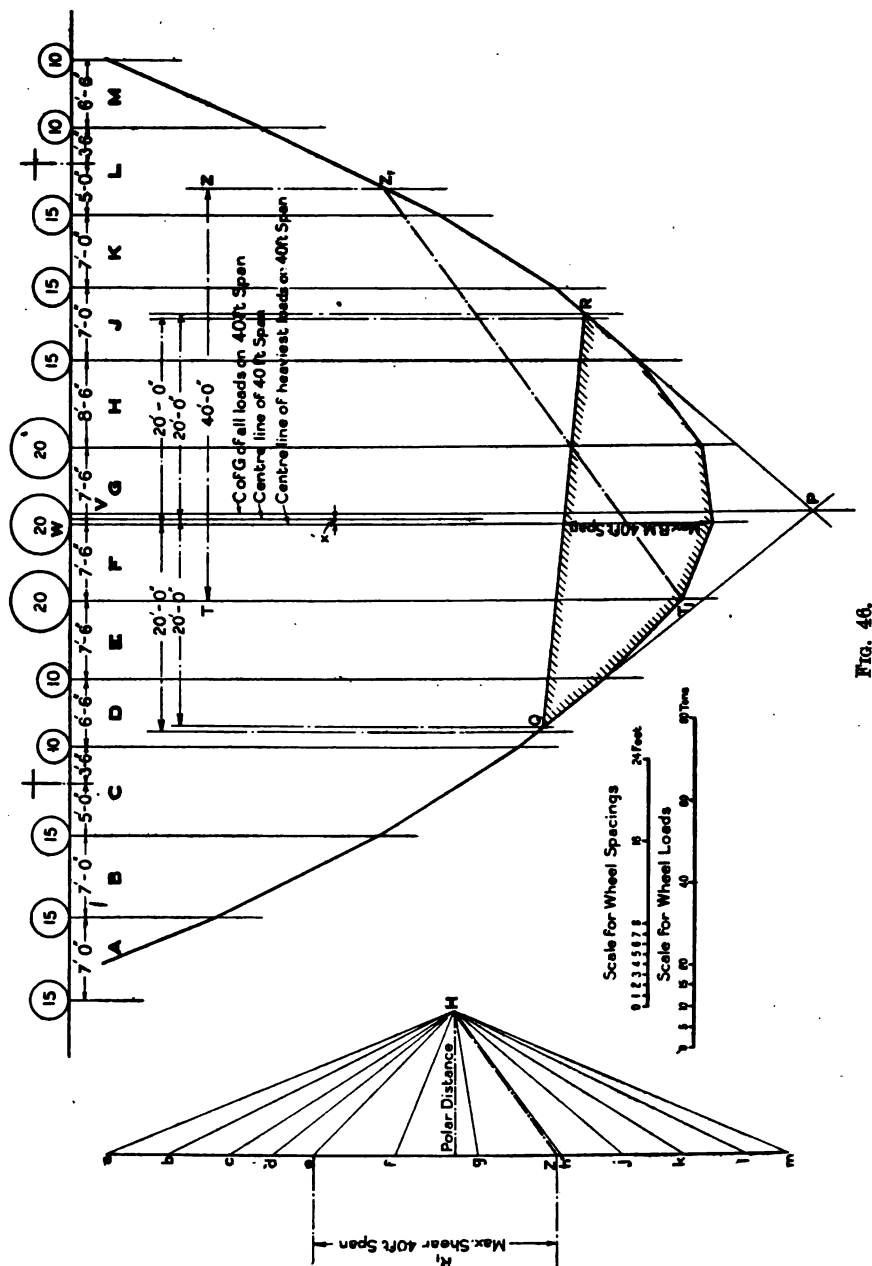


FIG. 48.

in order along this line, starting with the extreme left-hand axle load at the top. (A suitable scale for this would be 1" = 20 tons.)

A link polygon is now drawn for the axle loads, and to do this a pole point H must be selected, and lines drawn from this to each point on the load line. The point H should be chosen opposite the middle of the load line, and at such a distance from it that the first line Ha makes an angle of about 70° with the horizontal, this being to secure a suitable depth in the link polygon.

The horizontal distance from H to the load line is called the "Polar distance," and should be a definitely measured distance as $1\frac{1}{2}"$, $2"$ or $2\frac{1}{2}"$, etc.

The link polygon is constructed by drawing vertical lines under each wheel, and then in the bays thus formed, drawing lines parallel to the corresponding lines in the polar diagram. Thus the line drawn in the bay B is parallel to the line Hb, the line in the bay C is parallel to the line Hc, and so on.

It is quite immaterial where the link polygon is commenced, or where it finishes.

Determination of C. of G. of Loads on Span.—The object of the link polygon is first of all to determine the centre of gravity of all the loads which come on the span of 40 ft., when the greatest axle loads are at about the middle of the span.

The load FG is taken as the middle of the 40 ft. span, and the ends can be found by setting off a distance of 20 ft. to the proper scale, on each side of this load as shown; it can now be seen exactly what loads will be included on the 40 ft. span.

The Centre of Gravity of all these included loads can now be determined by the usual method, *i.e.* produce the lines of the polygon in bays D and J (the bays in which the ends of the span are situated) until they meet at a point P, through P draw a vertical line to meet the base line at V. *V is the centre of gravity of all the loads on the 40 ft. span.* (For a proof of this see text-books on Applied Mechanics or Graphics.)

Principle of Diagram.—The Principle on which the remainder of the construction depends may now be stated. (For a proof the reader is referred to a text-book on Mechanics.)

The Maximum Bending Moment produced by a series of concentrated loads rolling over the span occurs under the largest load of the series, and when this load and the centre of gravity of all the loads on the span are equidistant from the centre of the span, or, put in other words, the maximum bending moment occurs under the largest load, and when the centre of the span occurs midway between that load and the centre of gravity of all the loads on the span.

It will be noticed that in the example which is being worked out, there is no *largest* load, as there are three loads of the same intensity. It is, therefore, necessary to apply the above construction to each of the two loads adjacent to the Centre of Gravity, but in general it will be found that the maximum

B.M. occurs under the wheel which is nearest to the point V, in this case the wheel FG.

Construction of B.M. Diagram.—The construction of the B.M. diagram for the span can now be proceeded with. The centre of the span is now taken at W, a point midway between FG and V. Distances of 20 ft. to the right and the left are set off as before, and vertical lines are drawn to meet the link polygon at points Q and R. Join QR. The B.M. diagram is now the portion of the link polygon below the line QR, and is shown shaded in the diagram.

Special Cases.—Occasionally it is found that by changing the centre of the span to the point W, a new load will be included on one side, and one left out on the other; in this case it is necessary to find afresh the Centre of Gravity of all the loads, including this new load, and leaving out the one not again included.

A new position will be obtained, but afterwards the construction is similar to that given above.

The shaded diagram is an actual diagram of B.M.'s for the loads in the given positions on the span.

Scale of B.M. Diagram and Measurement of Maximum B.M.—If the greatest vertical intercept on the shaded diagram is measured it will be found to occur under wheel FG, this, as explained before, is the point of maximum bending moment.

This greatest vertical intercept represents to a certain scale, the maximum bending moment produced by the series of rolling axle loads, and the distance (x) on the space scale gives its position with reference to the centre of the span.

The scale of the B.M. diagram obtained as shown above is—

$$1'' \text{ (of vertical intercept)} = (H \times s \times l) \text{ ft.-tons of B.M.}$$

where H = polar distance in inches.

s = number of feet represented by 1" of base line.

l = number of tons represented by 1" of load line.

Equivalent Uniformly Distributed Load.—Now that the actual maximum B.M. produced by the loads has been found, it is easy to calculate the uniformly distributed load which would produce the same maximum value.

If M = actual maximum B.M. as found above (ft.-tons).

W = equivalent uniformly distributed load (tons per ft. run).

L = span in feet.

$$\therefore M = \frac{WL^2}{8}; \text{ or } W \text{ (tons per ft. run)} = \frac{8M}{L^2}$$

The value of W is the equivalent uniformly distributed live load which would produce the same maximum B.M. as the actual series of concentrated loads produce. This maximum B.M. due to the U.D.L.L. will occur at the centre of the span.

Equivalent Uniformly Distributed Live Loads for Maximum Shearing Force.—The equivalent uniformly distributed live load which produces a maximum B.M. equal to that produced by the actual axle loads, does not produce the same Shearing Force as these latter do.

Another equivalent load must be found for S.F., and as this load is usually greater than that for the B.M. it cannot be neglected.

The maximum S.F. on a span over which a series of axle loads are passing, occurs at the abutment, and when the first of the large loads is just on the point of passing off.

As the S.F. is just equal to the reaction on the abutment at that time, it can be calculated by the principle of moments, but it is easily obtained from Fig. 46.

Referring again to that diagram the maximum S.F. for a 40 ft. span will occur when EF (the first large load) is just about to pass from the span. Taking EF as T, the left-hand end of a span of 40 ft., the right-hand end will be at the point Z, the 40 ft. being, of course, scaled off horizontally on the base line. Join $T_1 Z_1$ and draw a parallel line HZ in the load diagram at the left. The distance aZ would represent the reaction at the left-hand abutment if all the loads were on the span; as, however, the load EF is the first one on the span being considered, the reaction for this span will not include any loads above EF or e . The line R_1 thus represents the maximum left-hand reaction, and, therefore, the maximum S.F. for this span.

Equivalent Uniformly Distributed Live Load for S.F.—Now the actual maximum S.F. produced by the axle loads has been obtained, the equivalent uniform load which would produce the same maximum can be calculated.

If S = actual maximum S.F. (tons).

l = span in feet.

W_s = equivalent uniformly distributed live load for S.F. (tons per ft. run).

$$S = \frac{W_s \times l}{2}; \text{ or } W_s = \frac{2 \times S}{l} \text{ (tons per ft. run).}$$

Values for Other Spans.—The same process has to be gone through for different spans, using the same link polygon each time, but the method of working is the same for each.

When all the values have been found a table can be constructed showing,

for each span, the values of the equivalent uniformly distributed live loads for maximum B.M. and maximum S.F. respectively, for the type locomotive chosen. Then from the table, curves, giving values of both loads for intermediate spans, can easily be constructed. A table and curves for the type locomotive adopted in Fig. 46 are given on pages 55 and 56.

It is the practice with some Engineers to add a percentage to the values actually found, in order to allow for future increases of axle loads. In place of this the author would suggest the adoption of a "type" locomotive, in which the loads used were the maxima ever likely to be placed on the various axles.

The working of the diagram is simplified by this method, as round numbers, both for loads and spacings, can easily be chosen.

Two Concentrated Rolling Loads.—When the rolling load consists of two axle loads only, as it does on a road bridge, where the live load is a traction engine or steam-roller, the same method as is described above can be used, the diagrams being much simpler in this case. On a bridge of moderate span, the traction engine and two loaded trucks can be taken for the live load, which raises the number of concentrated axle loads to six. In these spans it is not necessary, as a rule, to calculate the equivalent uniform live load; the maximum B.M. is sufficient for the purposes of design, but the uniform load can, of course, be obtained if required.

Special Loads.—There are occasional loads which have to pass over bridges, and which produce greater bending moments than the "type" load of the regular traffic. Such loads are breakdown cranes on railway bridges, or boilers and heavy machinery on road bridges in manufacturing districts. As the speed of these heavy rolling loads is comparatively low, most Engineers regard the impact allowance for the regular load as covering these special loads. There can be very little urged against this, but, nevertheless, some designers make a special allowance for these loads.

General.—It must be clearly understood that the diagram just constructed only gives the value of the absolute maximum B.M. produced by the given series of loads on the span.

It does *not* give the value of the maximum B.M. produced by the loads at intermediate points on the span. In order to get these, and thus to be able to draw a diagram of *maximum* bending moments for each point on the span, further diagrams would have to be constructed on the same principle, and by the same rule as that already given. Various other methods will be found in the articles referred to later; they are of hardly enough general interest to be included here.

The diagram of *maximum* bending moments produced by a uniformly

distributed rolling load is a parabola, and if a similar diagram were constructed for the axle loads it would be found that the latter would not be completely enclosed by the parabola, thus indicating that at certain points on the span, the maximum B.M. produced by the rolling loads is greater than that produced by the distributed load. Fig. 47 shows a typical case.

The excess of B.M. produced at these points in this way depends on the

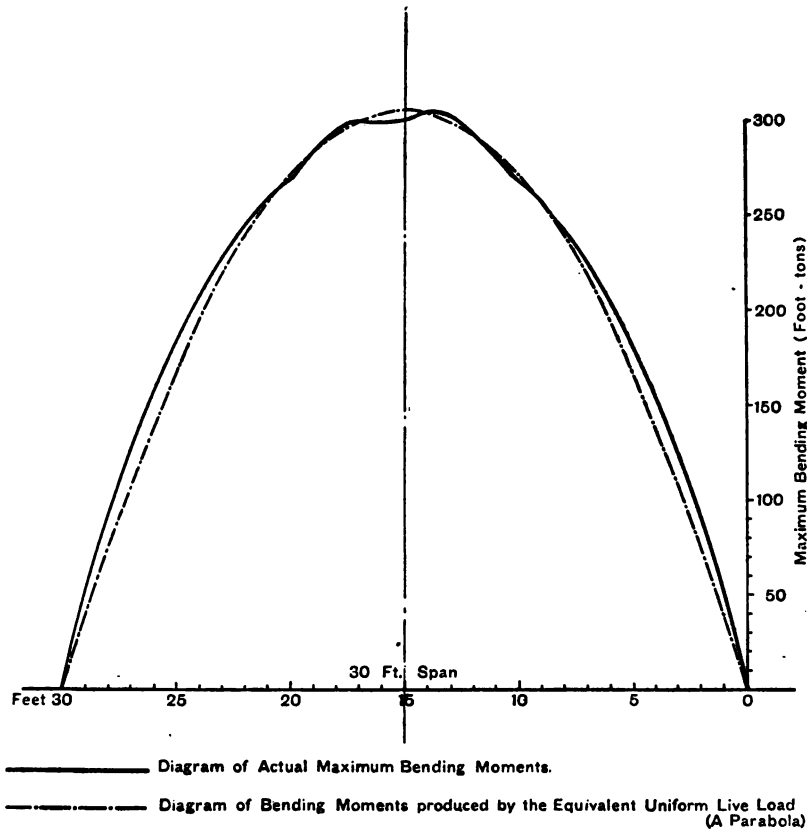


FIG. 47.

span, loading and axle spacing, it varies from about 10 per cent. excess on a span of 25 ft. to about 2 per cent. excess on spans of 60 ft. and over.

This excess effect must be carefully borne in mind when designing the details of the flanges. (See Examples I., II., and III., Chap. IX.)

It is allowed for by lengthening the second and, if necessary, the other tiers of flange plates over and above their calculated lengths—this is a common practice of designers, although the reason for it is often quite unknown.

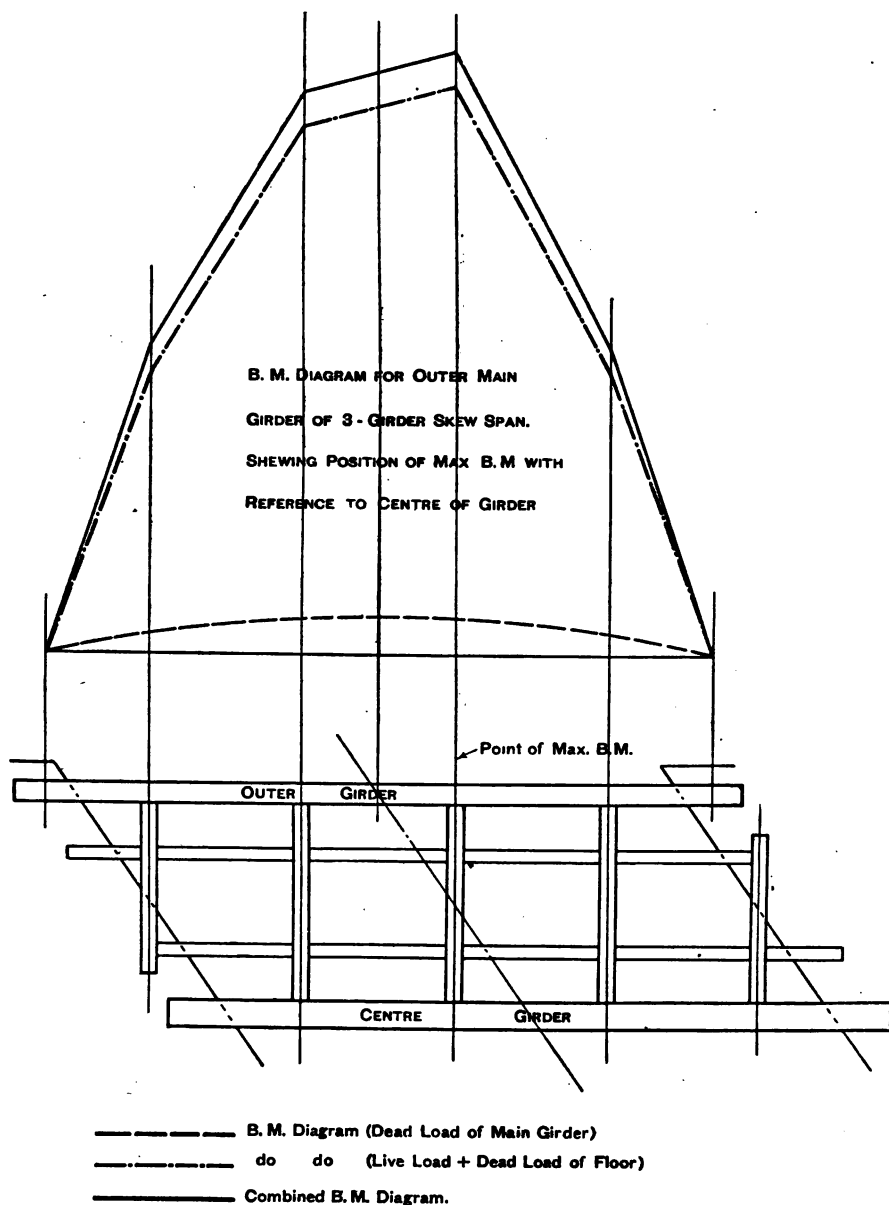


FIG. 48.

There is a similar excess effect in the case of the S.F., but owing to the practical methods of design adopted, it is not of such great importance.

Skew Bridges.—Tables of uniformly distributed live loads should not be applied to skew bridges, where there is a considerable angle of obliquity.

Owing to the unsymmetrical loading the point of maximum B.M. is often some distance from the centre of the girders.

The only plan which can be adopted in these cases is to draw a diagram of actual maximum B.M. for the span. The actual maximum B.M. will be produced at or near the centre of the clear span under the load, as in the case of a square span, but this point will not correspond to a similar point on the girder (see Fig. 48).

Allowance must be made for this difference when the detailed design of the girder is being considered, as the flange plates will not be symmetrical about the centre of the girder.

References.—For an interesting paper on the effects of axle loads and their spacings on the equivalent uniform load the reader is referred to the "Proceedings of the Inst. C. E.," vol. clxxvi.

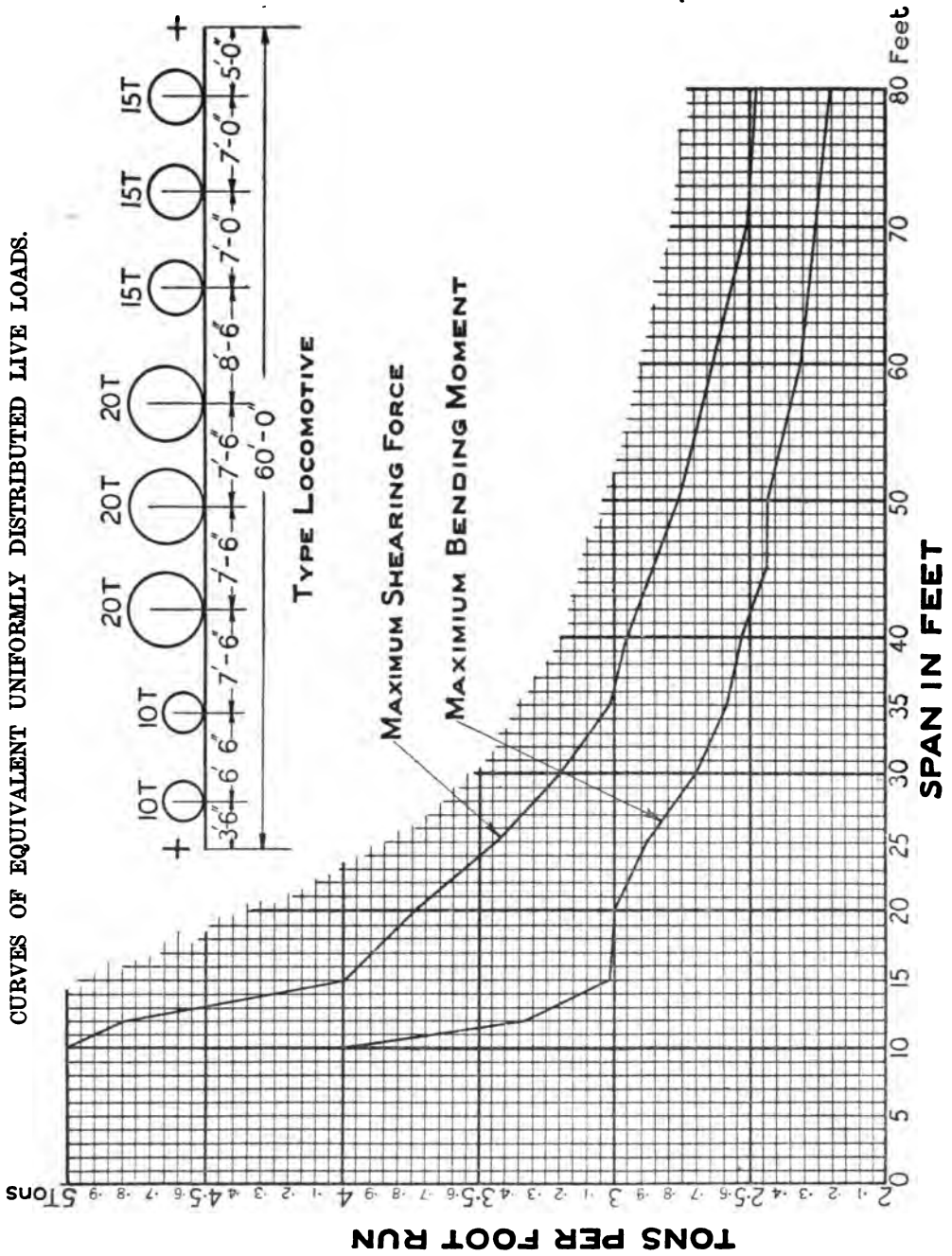
Also, further information about the diagrams used and described in this chapter can be obtained by reference to *Engineering*, vol. lxxxii., September 7th, 1906.

Other reference to the subject of live loads and equivalent uniform loads on bridges will be found in the "Proceedings of the Inst. C. E.," vol. cxli.

TABLE OF EQUIVALENT UNIFORMLY DISTRIBUTED LIVE LOADS FOR A SINGLE LINE OF RAILWAY DUE TO TYPE LOCOMOTIVE.

Span in feet.	Loads in tons derived from—	
	Max. B.M.	Max. shears.
10	40	50
12	40	55
15	45	60
20	60	75
25	72	86
30	81	96
35	91	106
40	101	118
45	110	129
50	122	138
55	130	148
60	139	158
65	148	167
70	158	176
75	167	187
80	175	198

CURVES OF EQUIVALENT UNIFORMLY DISTRIBUTED LIVE LOADS.



CHAPTER VI

RIVETS AND RIVETED JOINTS

A RIVET is a round bar of material used to permanently connect a piece of plate or bar to another piece adjacent to it. The rivets are placed transversely with the plates to be connected (Fig. 49), and keep the plates together

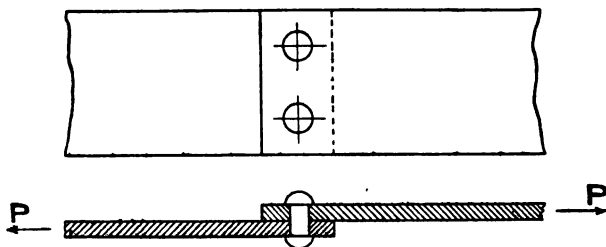


FIG. 49.

by their resistance to shearing, if the pull on the plates is axial, and by the projecting heads if the plates tend to separate laterally. Rivets are rarely used for this latter purpose, and are chiefly employed to connect plates subjected to an axial pull as shown in the figure.

An ordinary rivet as sent out for use in girder work is of the form shown in Fig. 50. It consists of a conical "snap" head, and a cylindrical "shank," and a rivet of this kind is described as a snap-headed rivet. The diameter of the rivet is measured

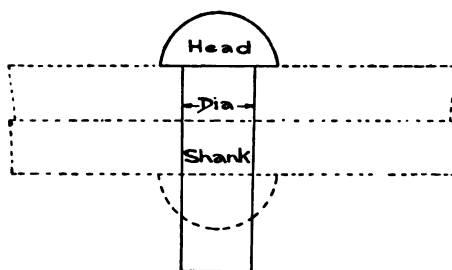


FIG. 50.

just under the head, as shown; owing to the action of the dies in forming the head the shank is slightly tapered. A $\frac{7}{8}$ " diameter rivet is formed from a round bar of steel $\frac{3}{4}$ " diameter; the shank will then be $\frac{3}{4}$ " diameter at the point, but it will be found to measure $\frac{7}{8}$ " under the head.

The shank of the rivet must be long enough, not only to pass through the

thickness of plates, but to leave enough metal projecting to form the other head. This head is formed either by hand, pneumatic or hydraulic tools, after the rivet has been heated to a welding heat and put into position in the hole. The length of the shank required to form the head, *i.e.* the amount of projection of the shank from the hole before the final riveting, depends on the diameter of the rivet and the thickness of plates passed through, and is termed the "allowance." The following table gives the average allowances required in ordinary work :—

Diameter of rivet.	$\frac{1}{4}"$	$\frac{1}{2}"$	$\frac{3}{4}"$	1"
Allowance (hydraulic)	$1\frac{1}{4}"$	$1\frac{3}{8}"$	$1\frac{1}{2}"$	$1\frac{3}{4}"$
„ (hand or pneumatic)... ..	$1\frac{1}{8}"$	$1\frac{1}{4}"$	$1\frac{3}{8}"$	$1\frac{5}{8}"$

The allowance depends somewhat on the number of thicknesses of material passed through by the rivet.

When a flush surface is required after riveting, "countersunk-headed" rivets are used (Fig. 51). Where possible, however, these flush-heads are formed in the final riveting and ordinary snap-headed rivets are used. This, of course, is only possible where one head is to be a snap one; if both surfaces have to be flush, then countersunk-headed rivets are a necessity. Other types and heads are used, but they are not common in girder work.

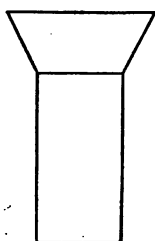


FIG. 51.

Types of Riveted Joints.—A. *Lap Joints.*—Lap joints are of several different types, but they are all similar, in that the rivet only passes through two plates, and that the failure of the rivet by shearing over one section brings about the failure of the joint. Such a rivet is said to be in *single shear*, as in Fig. 52.

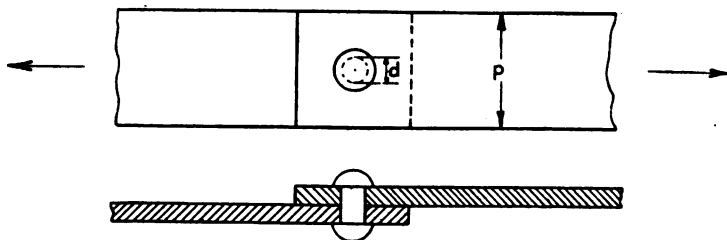


FIG. 52.

Suppose a lap joint is considered, having one rivet connection, and let the width of the strip p be equal to the pitch of the rivets. Then if this strip is part of the joint shown in Fig. 53, it is evident that the total pull

along this strip has to be resisted by the shearing resistance of the rivet. If, however, the rivet, of diameter d were very large, the metal left, $p-d$ as the effective width of the strip, would be very small, and the joint would fail by the plate tearing instead of the rivet shearing. The most economical joint would be the one in which the tendency of the plate to tear is just equal to the tendency of the rivet to shear. But if this condition is satisfied it will be found that the width p , or the pitch of the rivets, is much smaller than the pitches used in structural or, indeed, in any work, consequently, it is only necessary to consider the shearing resistance of the rivet in designing a joint of this type. The theoretical pitch would not be an economical one. But the shearing resistance can be increased without decreasing the tearing resistance by adopting the joint of which Fig. 54 is a type. This is called a double-riveted joint.

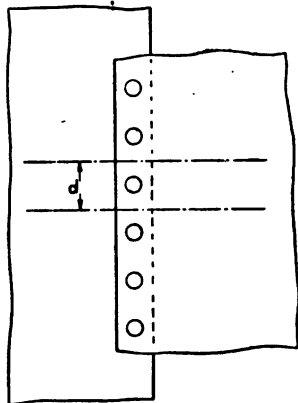


FIG. 53.

Before this joint can fail two rivet sections have to shear ; it is thus twice as strong as regards shear resistance, as the joint shown in Fig. 52, while the resistance of the plate to tearing is unaltered.

This is the principle adopted in all designs for riveted joints in structural work ; the tie-bar section is designed to carry the required load, this is the

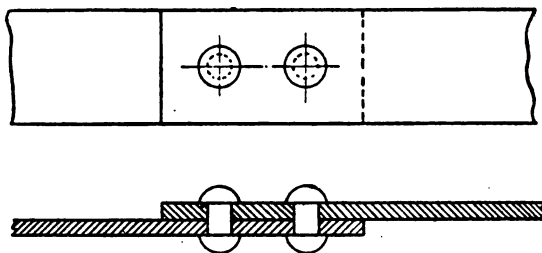


FIG. 54.

net section, and, if it is decided that there will be 4 rivets in line at any one section, the gross area can soon be found. Then, as the shearing resistance required is equal to the total load on the bar, the number of rivets can be calculated.

B. Butt Joints.—In a butt joint (Fig. 55) the plates to be connected are butted together, and are joined by means of one or two covers known as butt-straps ; two straps are generally used. The chief difference between

this type of joint and a lap-joint is, that before the joint can fail by shearing, each rivet has to be sheared in two places (at a and b), and such a rivet is said to be in *double shear*. The resistance of the joint to tearing is, in general, the same of that of the corresponding lap-joint.

The thickness of each of the butt-straps is made 75 per cent. of the thick-

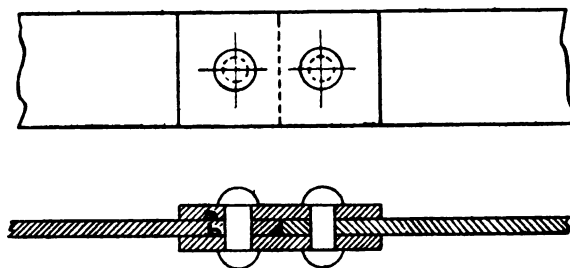


FIG. 55.

ness of the main plates, to obviate any possibility of the joint failing by these tearing.

The strength of a rivet in double shear can be taken at twice that of a rivet in single shear, although some authorities only allow it to be taken at $1\frac{1}{2}$ times. There is, however, no reason to suppose it is not equal to twice the strength of a rivet in single shear.

Bearing Values of Rivets.—If a large diameter rivet passes through a comparatively thin plate (Fig. 56), it is evident that the joint may fail by the

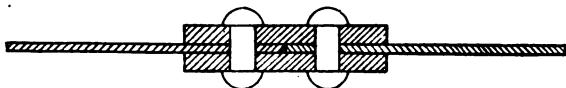


FIG. 56.

thin plate tearing out from the rivet in much the same way as a sheet of paper can be torn from a number of sheets fastened with a paper fastener.

To obviate the possibility of this occurring, the bearing value of the rivet must be considered. A limiting bearing stress or bearing pressure between the rivet and the plate is taken, and the bearing value V of the rivet in the plate is found from—

$$V \text{ (tons)} = fc \times d \times t$$

where

fc = maximum bearing stress (tons per sq. in.) ;

d = diameter of rivet ;

t = thickness of plate.

A usual value of f_c is 12 tons per sq. in., or about 50 per cent. greater than the allowable tensile stress.

If this bearing value works out lower than the shearing value, either single shear or double shear values, the bearing value must be taken as the maximum load which can be put on the rivet. The bearing value can be increased by increasing the thickness of the plate or the diameter of the rivets, and this has often to be done in practice with the web plates of plate girders, as will be seen later.

Lining of Rivets.—If the rivet in a riveted joint were placed too near the edge of the plate through which it passes, the joint may fail by the rivet shearing out the shaded piece of the metal or by cracking the plate as shown in Fig. 57. These methods of failure can be guarded against by the practical rule of making the distance l from the edge of the plate to the centre line of the rivet holes, called the "Lining" of the rivets, equal to at least one and a half times the diameter of the rivet. Occasionally it is only possible to have $l = 1\frac{1}{2}d$, but this is an absolute minimum. Tables giving the Standard lining for angle, tees, and other bars will be found in the Appendix.

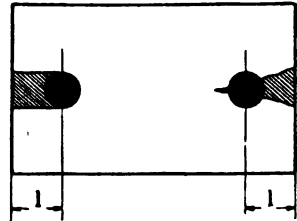


FIG. 57.

Pitch of Rivets in Structural Work.—As previously stated, it is not found economical in designing riveted joints to use the pitch of rivets which would make the joint of the greatest theoretical efficiency. This theoretical pitch would, in some cases, be so small as to render the "snapping" of the rivet holes impossible as they would be so close together. The general pitch of 4" is almost universal for structural work, except, of course, in special positions, such as the ends of girders, where the pitch of the rivets has to be determined by calculation. As the shearing force in the main angles of a girder decreases towards the centre of the girder, the pitch of the rivets could be increased in proportion, but this is not often done for two reasons. First of all, the frequent change of pitch renders the manufacturing costs higher, while with comparatively large pitches, the plates or bars tend to gape open and allow moisture to get between the plates. From these considerations 4" has been adopted as the standard pitch, and this is only departed from where the shearing force on the rivets renders it necessary, and where stiffener and other connections make it necessary to break the pitch.

Working Stresses in Riveted Joints.—The question of working stresses has been discussed in detail in Chapter IV., and the stresses recommended there have been used in the following examples of design :—

DESIGNS OF TYPICAL RIVETED JOINTS

Type No. 1. A Butt Joint in a Tie-bar.—A simple example of this type will be considered first, and although it does not apply directly to plate-girder design, it is a good example for elucidating the principles involved in the design of the later types.

Data.—Load in tie-bar	...	...	...	20 tons.
Working stress in tension	...	...	...	$7\frac{1}{2}$ tons per sq. in.
" " shear	...	...	...	$5\frac{1}{2}$ " " "
" " bearing	...	...	...	12 " " "
Net area of tie-bar required = $\frac{20}{7\frac{1}{2}} = 2.67$ sq. in.				

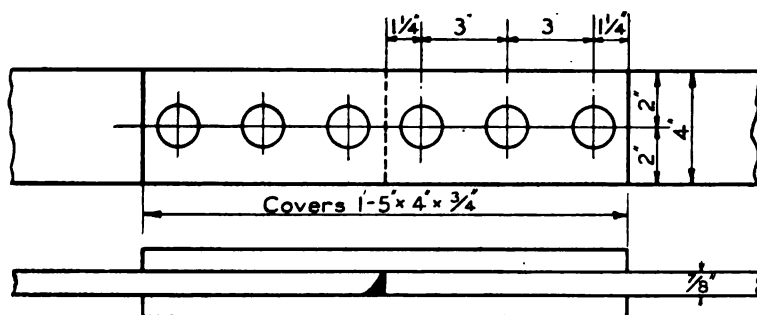


FIG. 58.

If rivets are taken at $\frac{7}{8}$ " dia. and the bar at $\frac{7}{8}$ " thick,

$$\text{Gross area required} = 2.67 + \left(\frac{15}{16} \times \frac{7}{8} \right)$$

(the dia. of a rivet hole is usually taken at $\frac{1}{16}$ " larger than the nominal rivet diameter) = $2.67 + 0.82$
= 3.49 sq. ins.

$$\therefore \text{Width of bar } \frac{7}{8}" \text{ thick} = \frac{3.49}{\frac{7}{8}} = 4 \text{ ins.}$$

A bar 4" wide $\times \frac{7}{8}$ " thick would thus be sufficient.

Riveting.—One $\frac{7}{8}$ " dia. rivet in double shear has a safe shearing resistance of $2 \times \frac{\pi d^2}{4} \times 5\frac{1}{2} = 6.6$ tons.

Bearing Value.—The bearing value of a $\frac{7}{8}$ " rivet in a $\frac{7}{8}$ " plate at 12 tons per sq. in. is $\frac{7}{8} \times \frac{7}{8} \times 12 = 9.2$ tons.

The shearing value 6.6 tons is, therefore, the criterion of strength in this example.

$$\therefore \text{Number of rivets required } \frac{20}{6.6} = 3 \text{ rivets}$$

The joint is shown in Fig. 58.

It should be noticed that the number of rivets should be sufficient to carry the full load which the bar is capable of carrying, irrespective of the actual *working* load on the bar. For example, the net area of the above bar is just equal to carrying the 20 tons which was to be provided for, but this is not always the case; often a bar will be found capable of taking much more than its ordinary working load.

Type No. 2. Joint in a Single Flange Plate of a Plate Girder.—

Data.—Flange plate: $12'' \times \frac{5}{8}''$.

Working stresses as before. Rivets $\frac{7}{8}''$ dia.

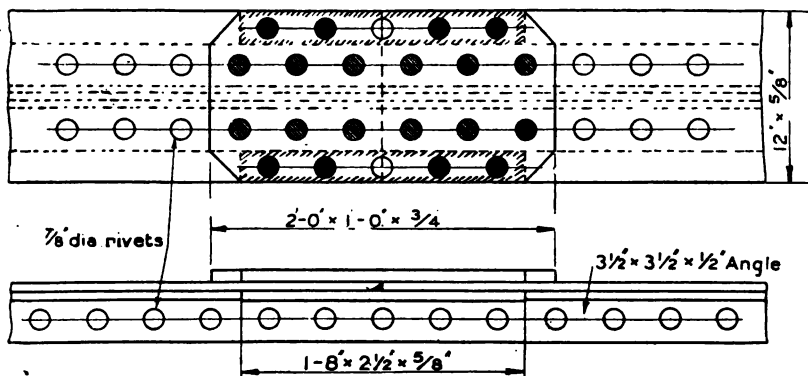


FIG. 59.

It is assumed that 4 rivet holes $\frac{5}{8}''$ dia. have to be subtracted in order to get the gross area.

$$\begin{aligned} \text{Gross area of plate} &= \left(12 - \frac{4 \times 15}{16}\right) \frac{5}{8} \\ &= \frac{33}{4} \times \frac{5}{8} = \frac{165}{32} = 5.17 \text{ sq. ins.} \end{aligned}$$

This flange can, therefore, carry a load of $5.17 \times 7\frac{1}{2} = 38.7$ tons.

The cover plate for this flange plate will be as shown in Fig. 59, and two cover strips will also be put in on the underside between the edges of the angles and the edges of the flange plates. As some of these rivets will be in

double shear and some in single shear, it is best to find how many single rivet areas $\frac{7}{8}$ dia. are required in order to carry a load of 38.7 tons.

Each area has a Shearing Value of $\frac{\pi d^2}{4} \times 5\frac{1}{2} = 3.3$ tons, and the Bearing Value of a $\frac{7}{8}$ rivet in a $\frac{5}{8}$ plate is at 12 tons per sq. in. $= \frac{7}{8} \times \frac{5}{8} \times 12 = 6.6$ tons.

The Bearing Value has rarely to be considered in joints of this kind, as except in exceptional cases it is always greater than the single shear value, and most of the rivets in the joint being designed, will be in single shear.

$$\therefore \text{Number of rivet areas required} = \frac{38.7}{3.3} = 12$$

The diagram (Fig. 59) shows the joint as completed, assuming the angles are $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{1}{2}"$.

It should be noticed that the rivets through the short cover strips are in double shear, and thus count two sections.

The covers are $2\frac{1}{2}" \times \frac{5}{8}"$, and their net area will be $(2\frac{1}{2} - \frac{1}{2}) \times \frac{5}{8} = 0.82$ sq. in., and they will thus carry a load of $0.82 \times 7\frac{1}{2} = 6.15$ tons.

They will thus require $\frac{6.15}{3.3} = 2$ rivet sections, *i.e.* 1 rivet.

Thus 1 rivet on each side of the joint would resist all the load the bar could safely carry in tension, but to provide a better cover for the joint, these small cover strips are frequently extended as shown above. They should always be used, if only to prevent moisture getting in the joint of the main flange plate, and they have in addition a certain stiffening effect on the joint.

In Fig. 59 the short cover strips are shown hatched, the rivets in single shear are shaded, and those in double shear are blackened. There are thus 6 rivet sections in single shear and 4 in double shear, making 14 altogether, as against the calculated number 12.

Type No. 3. Grouped Joint in Flange of a Plate Girder.—Considering next the jointing of the flange plates of a girder having 3 tiers of plates, but the joint may be a "shop" one, that is, the girder can be riveted-up complete before despatch. In cases of this kind, as explained elsewhere, the separate flange plate joints can be arranged to break joint with each other.

Suppose the plates to be jointed are $18" \times \frac{7}{8}"$, $18" \times \frac{7}{8}"$, and $18" \times \frac{3}{4}"$, and that the main angles are $6" \times 6" \times \frac{5}{8}"$, as shown in Fig. 60.

To get the net section of each flange plate the diameters of 6 rivets must be subtracted from the width.

$$\therefore \text{Net area of } 18" \times \frac{7}{8}" \text{ plates} = \left(18 - \frac{6 \times 15}{16}\right) \times \frac{7}{8} = 10.85 \text{ sq. ins.}$$

$$,, \quad ,, \quad 18" \times \frac{3}{4}" \text{ plates} = \left(18 - \frac{6 \times 15}{16}\right) \times \frac{3}{4} = 9.28 \text{ sq. ins.}$$

The respective carrying loads are 81.4 tons and 69.6 tons respectively.

The method of designing this joint is best seen by considering how the joint would fail by shearing, which is the method which has to be chiefly considered in the design, the joint being supposed to be of equal strength in all ways, it is immaterial which particular mode of failure is taken. The

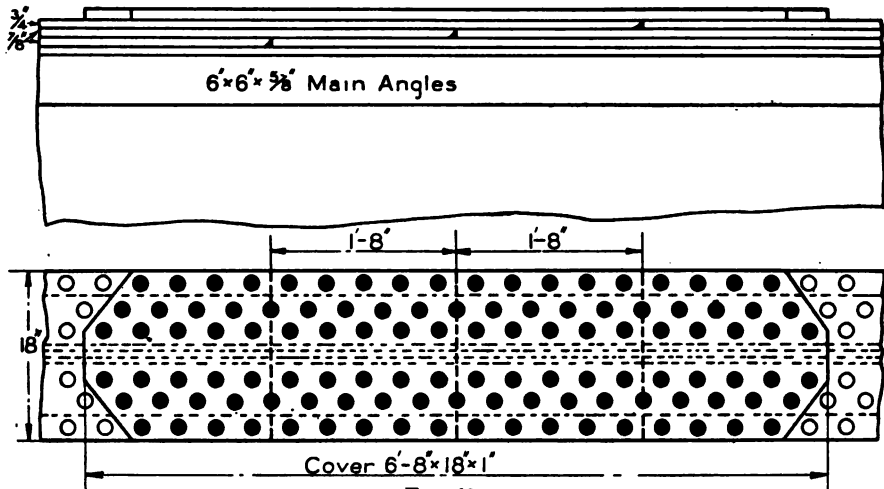


Fig. 60.

diagram (Fig. 61) shows failure by shearing, and it will be noticed that this takes place when the rivets from A to B, B to C and C to D have been sheared along *one* section only, therefore the design is a question of having sufficient rivet area in single shear, between A and B to carry the load on the bottom flange plate, between B and C for the load on the middle plate, and between

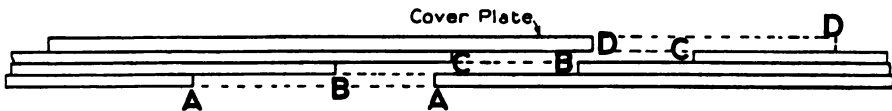


Fig. 61.

C and D for the load on the top plate. Bearing values need not be considered, as they are always greater (except in very special cases) than single shear values.

Then the area of metal in the flange at any joint must not be weakened by that joint, so the cover must at least make up the required net section, that is, it must be at least as thick as the thickest flange plate of the tiers. In practice, however, the thickness of the cover plates is very generally made $\frac{1}{8}$ " greater than that of the thickest flange plate.

Therefore, in the case under consideration, the cover will be 1" thick.

The number of rivets required between the various points can now be calculated. Between A and B, the rivet area must be sufficient to carry the safe tensile flange load on the net section of the bottom plate. This is 81.4 tons, and the single shear value of a $\frac{7}{8}$ in. diameter rivet being, as before, 3.3 tons.

$$\text{Number of rivets required} = \frac{81.4}{3.3} = \text{say } 25.$$

The same number will be required between B and C as the flange plate is the same section.

Between C and D,

$$\text{Number of rivets required} = \frac{69.6}{3.3} = 21.$$

This number of rivets is, in each case, required on each side of the joint, consequently the cover has to be extended to the left of A a distance at least equal to AB, and to the right of C a distance at least equal to CD:

The arrangements of the rivets are shown in Fig. 60. A cover strip on the outside of the angles has not been shown in this case, as its effect would not be so great, proportionally, as in the last type. The joint in the lower flange plate at A could be covered by a stiffener at that point.

It will be noted that the number of rivets in the joint is in excess of those actually required, owing to the practical consideration of the disposition of these rivets.

Often, cases will be found where it is necessary, in order to include one extra rivet, to include others as well.

A case in point is the length of the cover in this example. The cover is best made symmetrical about the centre line, and in order to include a minimum of 25 rivets at the left-hand side, 28 have to be included.

Type No. 4. Joints in Angle Bars.—The joint in an angle-bar is made by means of a cover angle placed inside the main angle, as shown in the diagram. The net sectional area of the cover angle must be at least equal to that of the main angle, and there must be sufficient connecting rivets to prevent failure by shearing.

Example.—Angle $6" \times 6" \times \frac{3}{4}"$.

The sectional area of this bar is 8.44 sq. in., and taking off the areas of 3 rivets holes, which will be cut by a section, the net area is—

$$8.44 - (3 \times \frac{5}{16}" \times \frac{3}{4}) = 6.33 \text{ sq. ins.}$$

The net area of the cover must be at least equal to 6.33 sq. ins. The bent plate forming the cover will be 10" wide (average), and subtracting

3 rivet holes, its net sectional area will be $\{10 - (3 \times \frac{5}{8})t\}$, where t is the thickness.

For the example given t works out at 0.88, say $\frac{7}{8}$ ". A bent plate 10" wide $\times \frac{7}{8}$ " thick would be used to form the cover.

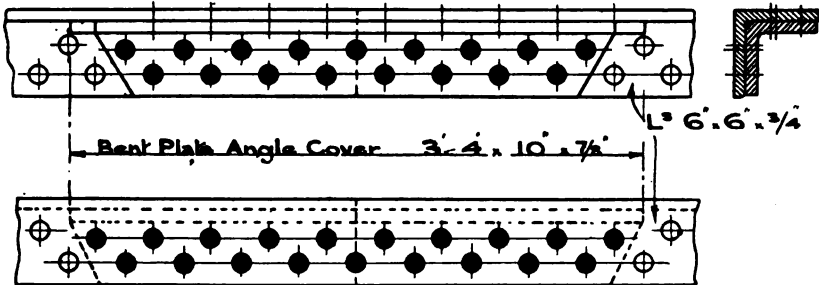


FIG. 62.

Number of Rivets.—Safe load on net area of angle at $7\frac{1}{2}$ tons per sq. in. = $6.33 \times 7.5 = 47.48$ tons.

The rivets must thus be capable of resisting this load.

One $\frac{7}{8}$ in. dia. rivet in single shear has a resisting value of 3.3 tons.

$$\therefore \text{Number of rivets required} = \frac{47.48}{3.3} = 14.4, \text{ say } 15 \text{ rivets.}$$

Fig. 62 shows the angle-joint separately, and not with the flanges and web plate.

It is always necessary in joints of this type to shape the ends of the covers

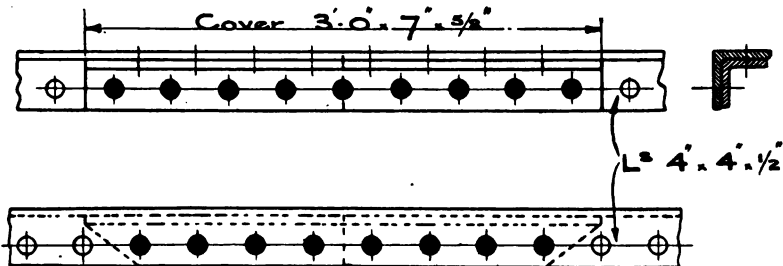


FIG. 63.

to avoid the rivets, as shown, and the lining of the rivets has also to be altered, as the ordinary lining leaves no room for the thick cover plate.

An example of the joint of a single rivet angle $4" \times 4" \times \frac{1}{2}"$ is shown in Fig. 63.

In these two examples single covers have been adopted, but in practice

all covers should be double, *i.e.* one on each side of the joint covered, which would bring the rivets into double shear, and, therefore, half the number would be required on each side of the joints in the examples described above.

Type No. 5. Web Plate Joints.—In a web plate joint, the shearing resistance of the rivets must be made equal to the shearing resistance of the net sectional area of the web. As the comparison is between two shearing

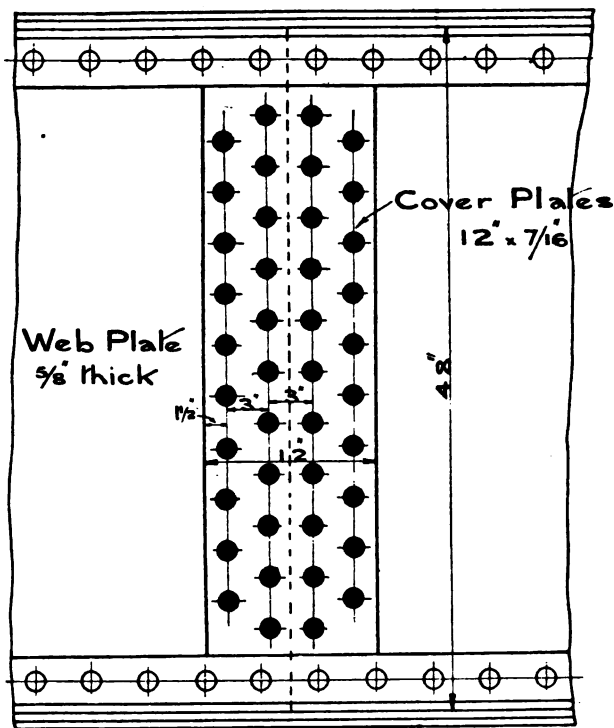


FIG. 64.

resistances, it follows that the total rivet area resisting shear must be equal to this net web sectional area.

The relation between the gross and net web areas cannot be found exactly until the design has been detailed, but usually the net area is taken at $\frac{3}{4}$ of the gross area (this is allowing for a 4" rivet pitch).

Double cover plates are used for web covers, so that the rivets in a web joint are in double shear.

Example.—Web plate, $48" \times \frac{5}{8}"$.

Net area of plate = $\frac{3}{4}(48 \times \frac{5}{8}) = 22.5$ sq. ins.

Each $\frac{7}{8}$ dia. rivet in double shear has an area of 1.2 sq. in., and the area of the rivet sections at the joint must be just equal to the net area of web at the joint.

$$\therefore \text{Number of rivets required} = \frac{22.5}{1.2} = 19.$$

The greatest number of rivets which could be put in a vertical section 48" long at 4" pitch is 12, and, therefore, a double-riveted web cover will be required. If, however, only 15 rivets were wanted, these could be got in a single-riveted cover using a 3" pitch.

The number of rivets used in web cover joints should not be stinted, as it must be remembered that, although the web is assumed to take vertical shearing force only, there is, in practice, a certain amount of horizontal tensile and compressive force taken by the web, and, therefore, by the web covers.

Following the usual rule, given before, for double cover plates, each web cover plate is made above $\frac{3}{4}$ of the thickness of the main web plate.

The design of the joint is shown in Fig. 64.

Type No. 6. Web Joint in which Part of the Web Section is assumed to take Bending Stresses.—When part of the web section is assumed to resist horizontal stresses due to bending, it is usual to take this part as equal to $\frac{1}{8}$ of the gross area of the web.

Example.—Web plate, 54" $\times$ $\frac{3}{4}$ ".

Net area of plate = $\frac{3}{4}(54 \times \frac{3}{4}) = 30.37$ sq. ins.

Area assumed to resisting bending = $\frac{1}{8} \times 54 \times \frac{3}{4} = 6.75$ sq. ins.

If we assume the flange stress as being $7\frac{1}{2}$ tons per sq. in., the web will thus carry a flange load of $6.75 \times 7\frac{1}{2} = 50.7$ tons. This load must, therefore, be carried over the web joint by means of riveted covers suitable for carrying tensile and compressive loads of this amount. These covers are placed across the web joint immediately alongside the main angles, and one on each side of the web plate.

Design of Covers.—As the web thickness is $\frac{3}{4}$ ", the covers must be made $\frac{7}{8}$ " thick, as it is necessary to get them as narrow as possible. Assuming two rivet sections to be deducted from their width d , their net area will be each $(d-2)\frac{7}{8}$, or for both of them $(d-2)\frac{7}{4}$. Now the horizontal stress on these covers cannot be taken at $7\frac{1}{2}$ tons per sq. in. (the flange stress), because of their position with reference to the neutral axis. If we suppose the stress in them to be approximately $5\frac{1}{2}$ tons per sq. in. the width d will be found from $\frac{7}{4}(d-2) 5\frac{1}{2} = 50.7$ tons, or $d = 7.3$, say $7\frac{1}{2}$ " wide.

The riveting for this cover has also to be designed with reference to the

neutral axis of the girder ; the outer line of rivets through the cover will take the greatest share of the load, and so the stress in them must not exceed the safe working stress in shear, viz. $5\frac{1}{2}$ tons per sq. in., or the bearing value of 12 tons per sq. in.

As the intensity of the stress varies with the distance from the neutral axis, a simple calculation will enable a mean stress for the cover to be found,

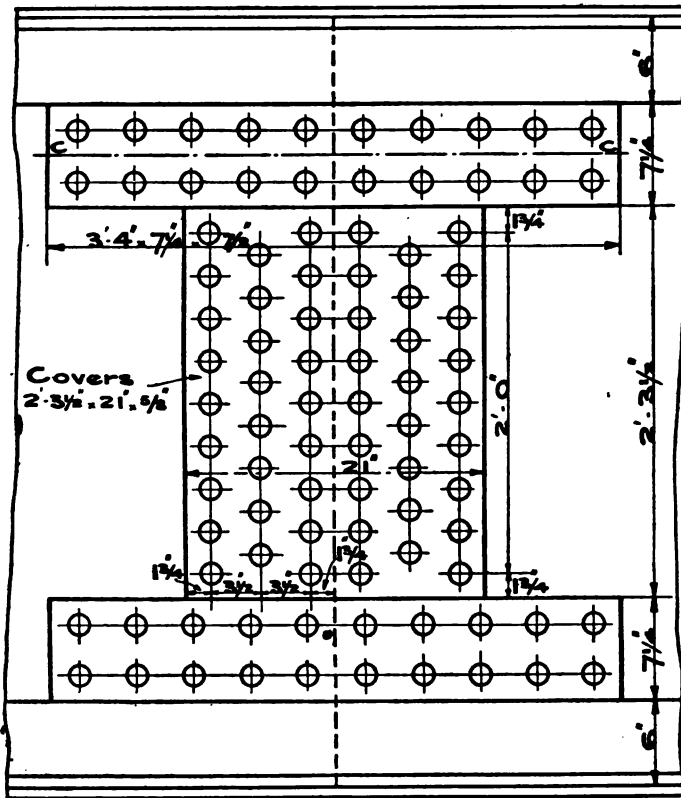


FIG. 65.

on the assumption that the maximum stress occurs at the outer line of rivets. The mean stress will, of course, occur at the horizontal centre line CC of the cover.

The above method of designing this type of joint is not theoretically an accurate one. To design the joint from theoretical considerations would require each rivet to be taken separately, and the result would not justify the time involved. This is one of the many cases in which an approximate rule can be substituted for the strict theory.

For the example under consideration, this will work out at about 4.9 tons per sq. in. for shear and about 10.5 tons per sq. in. for bearing.

Now the bearing value of one $\frac{7}{8}$ " rivet in a $\frac{3}{4}$ " plate at $10\frac{1}{2}$ tons per sq. in. is—

$$\frac{7}{8} \times \frac{3}{4} \times 10\frac{1}{2} = 6.9 \text{ tons.}$$

and in double shear at 4.9 tons per sq. in.

$$= 1.2 \text{ sq. in.} \times 4.9 = 5.9 \text{ tons.}$$

The latter value has to be taken for calculation

$$\therefore \text{Number of rivets required} = \frac{50.7}{5.9} = 10 \text{ rivets.}$$

These are spaced 5 in each row, and at 4" pitch, and each of the 4 covers will, therefore, be $3' 4" \times 7\frac{1}{4}" \times \frac{7}{8}"$.

Design of Web Cover.—This is designed precisely as in type No. 5, except that the available depth has been decreased, and the cover must, therefore, be wider to take in all the necessary rivets. This has the effect of making the rivet stresses unequal, but as the actual shearing force at a web joint is not such as to stress the plate to its maximum capacity, this effect is not very serious.

$$\text{Number of rivets required} = \frac{30.37}{1.2} = 25 \text{ or } 26.$$

The detailed design is shown in Fig. 65.

Design of Riveted Joints for Bracing Bars.—These joints usually consist in the jointing of angle or tie-bars to gusset plates, and of the latter to the flange plates of the main girders.

The connection of a tie-bar to its gusset is designed on the same principle as the joints already considered, viz. the shearing resistance of the connecting rivets is made equal to the tensile or compressive resistance of the bracing bar. Bearing values of rivets have rarely to

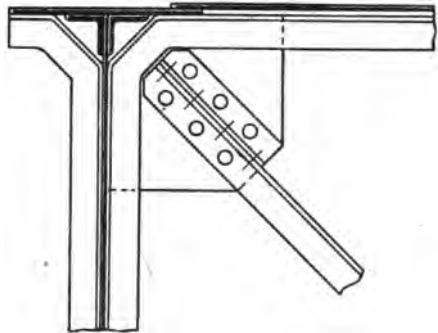


FIG. 66.

be considered, as the rivets are generally in single shear. In the case of the connection of an angle-bar to a gusset plate, both legs of the angle should be connected to the plate (the vertical leg by means of a cleat, Fig. 66). However, this need only be done in important connections; often

an angle-bar is used for bracing instead of a flat bar for the sake of the extra stiffness.

When connecting a gusset plate to a main girder (Fig. 67) the direction and magnitude of the forces tending to tear the plate away from the flange must be considered. By drawing a triangle of forces for the gusset plate it will be seen that there are two forces (at right angles to each other) tending to tear the gusset plate away. The connecting rivets must, therefore, be

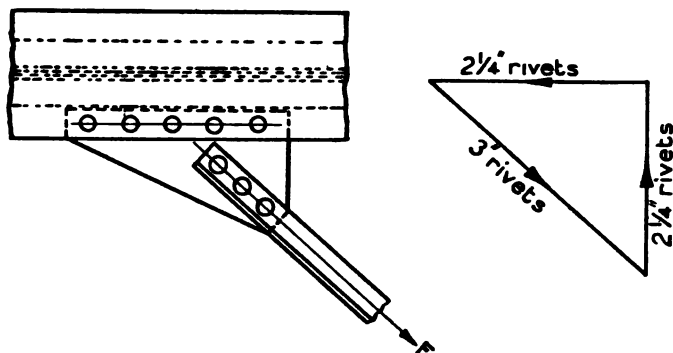


FIG. 67.

strong enough to withstand each of these shearing forces separately. Of course such a connection hardly warrants a calculation, but the underlying principle must be kept in view.

Also, so far as possible, the rivets connecting the gusset plates to the main girder should be distributed equally between the two sides of the centre line of the connecting bar, otherwise the rivets will be subjected to an additional twisting stress.

This is not always possible in small joints, owing to practical reasons, but it should be done whenever it is possible.

CHAPTER VII

THE PRACTICAL DESIGN OF PLATE GIRDERS

Economics of Design.—In designing constructional steelwork of all kinds, a good design depends not so much upon theoretical refinements as upon practical considerations. Theoretically, there is no such thing as a good design for a bridge, as many of the stresses are indeterminate and based upon assumption, which may or may not be correct; but, from a practical standpoint, a good design for a bridge is that which will not only safely carry its heaviest loads with a minimum of maintenance charges, but also one that is cheap in construction. Therefore, the chief aim of designers should be to produce designs giving the most efficient structure for the least capital cost.

This principle appears to receive little consideration from many purely designing Engineers, as is easily seen by a glance at the bridge designs sent as working drawings to manufacturers of bridgework.

Comparing two separate designs for a structure, of the same type and span, and to carry exactly the same loads, it is quite possible for one to cost 40 or 50 per cent. more than the other, and take twice as long to supply, and these variations are due to several important practical considerations.

The net manufacturing cost of bridgework, exclusive of fixed or common charges, on cost, etc., depends upon—

1. The average cost of the materials.
2. The cost of the labour or workmanship on the materials.

Taking two designs of the same total weight, there will be very little difference in the relative cost of the materials, and any saving that can be obtained under this head will be considered later, but with the cost for workmanship the case is totally different.

The amount of work required on the one design may be double that required on the other, then, if so, it follows that the labour costs are higher, and assuming the weights to be about equal, the rate per ton of the finished structure is correspondingly increased. The relative weights of two designs for the same bridge, therefore, offers no clue to the respective total costs ;

this can only be judged after a close inspection of the workmanship required on each. It is the actual labour or workmanship on the materials which influences the cost perhaps more than any other consideration.

Generally speaking, the one principle to be observed in economical designing is to make workmanship an absolute minimum for the weight of material required.

A reduction in the amount of workmanship does not by any means imply that the finished structure will be weaker, rougher, or less efficient in any way, as most of the workmanship which can be reduced is quite superfluous so far as "efficiency for duty" of the bridge is concerned.

The chief cause of this superfluous workmanship is due to striving after architectural effects, or some form of ornamentation to which mild steelwork does not lend itself. Except in very special cases, where appearance may

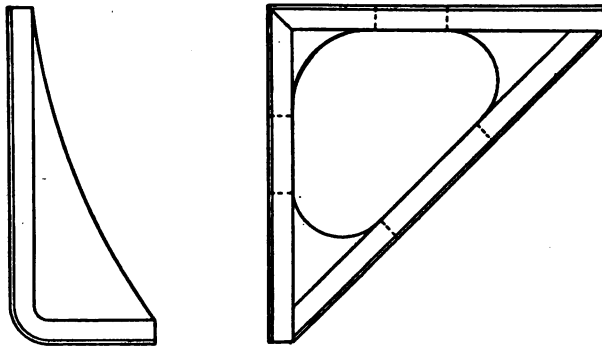


FIG. 68.

justify the extra workmanship and expense, all attempts at æsthetic effects should be studiously avoided.

There is, perhaps, no more fruitful source of heavy workmanship costs in bridgework than the use of curved lines, whether in the general outline or in the detail of designs. Curved gusset plates, tie plates, brackets, or plates with re-entrant angles (Fig. 68), are all undesirable in economical designing. Curved lines, whether convex or concave, are always expensive; there is no special machine for shearing to a curve, so that when curved lines are required, not only have the edges of the plates to be chipped and hand-dressed, and handwork is the most expensive of all workmanship, but a large amount of material is wasted, as a plate with straight edges must be ordered in the first instance.

Plates with concave edges are more expensive than those with convex edges, as the former have to be cut out of a larger plate by punching a series

of holes round the shape required, and the jagged edges then chipped or ground smooth by an emery wheel, altogether a very tedious and expensive process.

An excellent example of a design involving curved lines and extra labour costs, may be seen by comparing an ordinary rectangular plate girder with one of the hog-backed or fish-bellied type (Fig. 69) of the same span.

The extra workmanship required for the hog-backed or fish-bellied type is as follows :—

- (a) Bending flange plates, main angles, and covers to curve.
- (b) Chipping and dressing top edge of web plates.
- (c) Smithing stiffeners, which are of different lengths, and have to be smithed to bevel.
- (d) Extra template making due to top and bottom flanges, stiffeners, etc., being different.

Altogether this extra labour will cost considerably more per ton than the

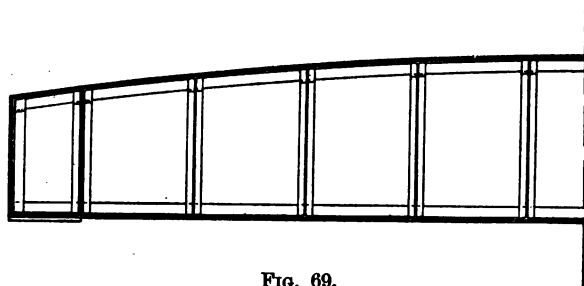


FIG. 69.

labour required on a rectangular girder, while the only saving, if any, will be in weight, and this is usually very small, and often negligible.

It is often claimed that hog-backed girders are more scientific in shape and of better appearance than plain girders; this may be so, but at the same time, they are certainly not as economical, so it is difficult to see how their adoption can be justified.

Girders with rounded ends (Fig. 70) often find favour with some Engineers, but they should always be avoided. Although such girders may look better and more finished in appearance, the question to be considered is whether the extra money spent on such refinements is conducive to economy and utility. In railway underbridges, girders between two lines of way are often made with rounded ends, the object being to reduce the resistance a square-ended girder might offer in the event of a derailment. There may be something said in favour of doing this if the tops of the girders are near rail level, but when they are several feet above rail level, as is usually the case,

there is certainly no advantage to be gained in employing girders with rounded ends.

Another factor greatly influencing the cost of workmanship on a bridge is the amount of smithing required. Smiths' work is always expensive, as it includes a great amount of slow and tedious handwork. In ordinary plate

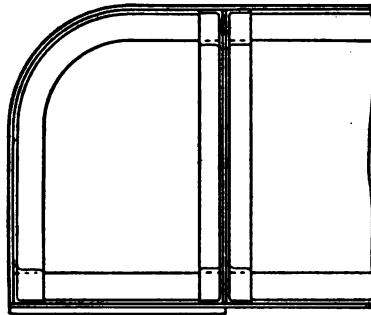


FIG. 70.

girder bridges there is usually no need for smithing beyond that required in bending stiffeners, and these should always be of the joggled or kneed type, as shown in Fig. 71.

Square knee stiffeners, as shown in Fig. 72, should be avoided, as they have to be cut and welded by hand, and there is always a liability to fracture

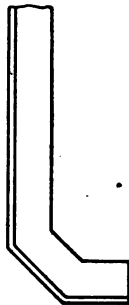


FIG. 71.

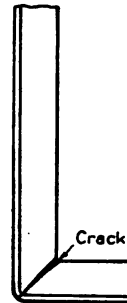


FIG. 72.

at the crank, especially in the case of tee stiffeners. Joggled and kneed stiffeners can be blocked in a hydraulic press, the former being the cheapest, but a square knee costs as much as both.

Bends to be made at the ends of long bars are extremely awkward to make, and the labour is costly.

A good example of excessive smithing and its elimination can be obtained

by comparing the two designs for a rail-bearer (Figs. 73 and 74). In the former it will be seen that each main angle requires two knee bends and four joggles (two each way), making four knee bends and eight joggles for each rail-bearer, which, of course, make a very expensive job. All this smithing

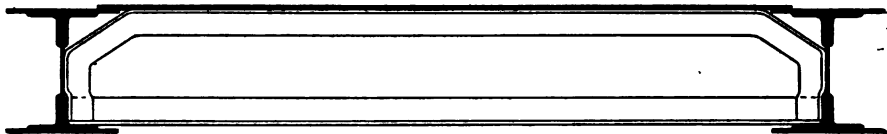


FIG. 73.

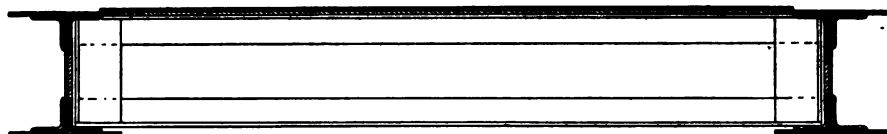


FIG. 74.

can be avoided by adopting the design shown in Fig. 74, in which smithing is entirely eliminated.

Another example of unnecessary smithing is sometimes seen in the end connections to trough flooring, where angles are bent to the shape of the troughs, as shown in Fig. 75. This intricate bending can be avoided by using

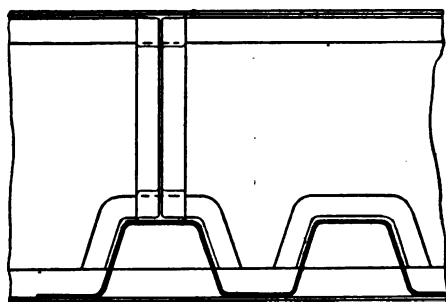


FIG. 75.

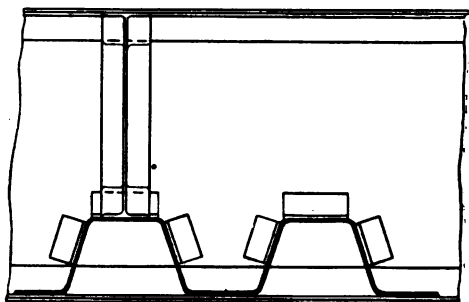


FIG. 76.

angle cleats (see Fig. 76), which is practically the same connection as before, only without the bends.

The degree of finish required also has an important bearing on the labour costs of steelwork.

Stiffeners are sometimes shown on drawings to have the corners cut off and filed up as shown in Fig. 77. This practice is sheer extravagance, as each stiffener done this way will cost 3s. to 4s. extra.

Angle cleats for rolled joist connections are sometimes required to be

shaped so as to fit against the top and bottom flanges, as in Fig. 78. There is no good reason for this, as cleats cut off square (Fig. 79) are quite as efficient, and do not cost half as much.



FIG. 77.

It is often specified that the edges of small gusset or connecting plates, and ends of kneed stiffeners, are to be planed, chipped or filed. This refinement is in many cases unnecessary, as sheared edges are quite as clean as planed edges, and there certainly cannot be any objection to the marks left by the cold saw, although occasionally there is, without good reason.

Material Sizes.—The choice of sections is a question also having an important influence on the economical design and manufacture of steel bridgework, and although not affecting the cost to such an extent as workmanship, it must, nevertheless, receive the closest attention of designers.

Structural steel sections have been standardised by the Engineering Standard's Committee; but for many of these sections there is little or no demand, and they are not frequently rolled, consequently, if designers use any of these latter, it may mean serious delay to the work through non-delivery of the material.

Manufacturers of bridgework are generally under a heavy penalty if the promised time for delivery as stated in the tender is exceeded; therefore, if

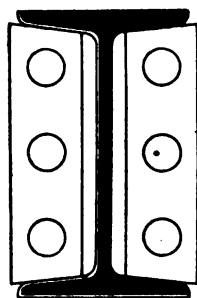


FIG. 78.

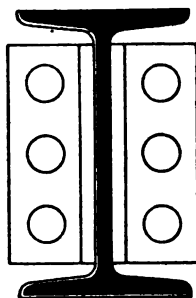


FIG. 79.

the work in hand is urgent, it is of the greatest importance to use sections that are easily obtainable and in common demand.

If odd sections that are not easily obtainable be adopted, and if the work in hand is urgent, then special arrangements may have to be made with the steel manufacturers for their supply, or larger common sections planed down to the required size.

Both of these expedients involve additional expense, usually to be borne by the bridge manufacturer.

If the rolling-mills are busy with orders for common sections, and steel manufacturers are asked to stop and supply a smaller order for other sections urgently required, then it necessitates changing the rolls, rather an expensive proceeding for a small order. If the cost of changing the rolls is say £30, and the order, say for 10 tons, then the basis price for the material is increased 60s. per ton, while for a larger order, say for 150 tons, the increased price would only be 4s. per ton. It is evident, therefore, that it is very uneconomical to adopt sections that are not easily obtainable, unless large quantities are required, which is unusual.

The planing down of larger sections which may sometimes have to be resorted to is a very wasteful and expensive method of obtaining the sizes required.

Designs for ordinary plate girder bridges may often be seen having many different sections, sometimes of curious sizes, *e.g.* $4\frac{1}{2}$ " by $3\frac{1}{2}$ " angles, or 5" by $3\frac{1}{2}$ " tees, and such like. Such odd sections should always, as far as possible, be avoided, and the number of different sections in a job kept down to a minimum.

There is no reason why the main angles and end angles should not be of the same size, and all the stiffener angles and tees of one size.

The sizes of sections most commonly rolled can be obtained from the useful section books issued by the various steel manufacturers.

When consulting these books, the designer should study the somewhat formidable list of extras which are charged for section, length, thinness, thickness, etc., as by a judicious selection of sizes many of these extra charges may be avoided, and the initial cost of the material reduced.

For convenience, a list of extras for sections, plates, etc., together with the rolling limits, is given in the Appendix.

COMPOSITION OF THE FLANGE SECTION

Practice is not uniform respecting the design of the flange section for plate girders. It is the general British practice to assume that the bending action is resisted entirely by the flanges, which are taken to comprise the flange plates and angles only, leaving the shearing action to be resisted wholly by the web, in each case assuming the stress to be uniformly distributed over the section. These assumptions are purely arbitrary. It is customary in America, and with a few of the leading British Engineers, to take one-eighth of the web plate in solid web girders as acting with the flanges in resisting tensile or compressive stresses. As stated in Chapter III., some designers consider that the flange section should consist of the flange plates and the

whole of the main angles, while others disregard the vertical limbs of the main angles. Again, in some offices, net areas, i.e. deducting rivet holes, are taken for both tension and compression flanges, while in others, it is usual to take the gross area for the compression flange and the net area for the tension flange. The girder will have a higher factor of safety if the horizontal limb only of the angles is included in the flange area, neglecting the vertical limbs and also the portion of the web between them, taking net areas of plates and angles for both compression and tension. On the other hand, the girder will be more economical in material if both limbs of the angles are considered in the flange section, together with the portion of the web between the vertical limbs, taking net areas in tension and gross areas in compression. It is purely a matter of individual opinion which method to adopt.

Width of Flange Plates.—The width of the flange plates is generally made a proportion of the span, and must be sufficient to prevent lateral buckling of the compression flange. A usual proportion is to make the width from one-thirtieth to one-fortieth of the span, but the width is sometimes restricted by questions of clearance.

Sizes of Main Angles.—The sizes of the main angles should be made as large as possible, consistent with the flange width, and it is good practice for at least one-third of the total flange area to be composed of angles, as not only are angles cheaper than plates, but large angles are also theoretically better than small ones.

The sizes of main angles used for ordinary girderwork range from 3" by 3" to 6" by 6"; larger sizes up to 8" by 8" can also be obtained, but these should never be used except in special cases.

In cases where the specification only allows the use of the horizontal limb of the angle, as is sometimes the case, unequal angles are frequently used, the longer leg being placed horizontal.

The sizes of the main angles depend to a great extent upon the diameter of the rivets connecting them to the web, as, should double riveting be necessary, the vertical leg of the angles must not be less than $4\frac{1}{2}$ " where $\frac{7}{8}$ " diameter rivets are used. A $\frac{3}{4}$ " diameter rivet single riveted requires a 3" by 3" angle, and a $\frac{7}{8}$ " diameter rivet, single riveted, a $3\frac{1}{2}$ " by $3\frac{1}{2}$ " angle as minimum sizes.

Flange Plates.—Flange plates should not be less than $\frac{3}{8}$ " nor more than $\frac{7}{8}$ " thick, intermediate thicknesses varying by sixteenths. Whenever possible flanges should be made up of plates all of one thickness. The total thickness of the flange plates and main angles should not be more than five times the diameter of the rivet employed, as the riveting (especially field riveting) is not satisfactory when the "grip" exceeds this proportion. In proportioning

the flange area there is nothing to be gained in trying to choose plates to exactly balance the area required, as although this may be the scientific method, it is in the majority of cases very unpractical.

Both tension and compression flanges should, where possible, be of equal widths and section as this is the cheapest and simplest method. No advantage is to be gained by making the top flange narrower than the bottom flange, as is often done, except in exceptionable cases. Edge angles are sometimes used when the width of the top flange is restricted, and these not only help to make up the section, but also stiffen the narrow flange against buckling, although at the same time the area is usually wasted towards the ends of the girders as can be seen from the bending moment diagram. It is the best practical method to keep both flanges of the same section; and this will be found—approximately at least—theoretically correct, as the ratio of the gross area to the nett area will be roughly that of the working stress in tension to the working stress in compression. In many cases with spans about 30 ft., one flange plate is ample, and when this is so an edge-rolled section—commonly called a flat—may be used instead of an ordinary plate, which has to be sheared, and in a great many cases planed.

Although flats cost more than plates they have the advantage of saving the cost of shearing and planing, and quicker delivery from the mills is generally obtained. Flats have the further advantage of “weathering” better than plates, i.e. the edges do not invite oxidation so readily as a sheared or planed edge, owing to the rolling skin on the edge not being removed. Flats can be easily obtained in sizes from an even inch in width up to 21”, and of varying thicknesses. Above 18” wide it is generally better to use plates, and, in any case, flats should never be used where the flange consists of more than one bar in one single length without joint, and then only provided that they are perfectly straight, having cleanly rolled and square edges.

The question of sheared versus planed edges for flange plates may be discussed here. Practically all Engineers specify that all edges of plates and bars must be planed, and that no rough edges fresh from the shears will be permitted. The cost of planing is always heavy, as the handling and setting of long plates in the machine occupies a good proportion of the time. Shearing can be done so neatly with the powerful tools now in use that it would be very difficult to distinguish between the two kinds of edge at a distance of a few feet away, and for this reason it is often considered that planing is superfluous, especially for work going to some comparatively undeveloped country, or to be placed in isolated positions, as is often the case. Planing has the advantage of ensuring that the edges of the plates

are kept truly flush with one another along their entire length, and that the whole of the surfaces are in contact, preventing any possibility of moisture getting between the various tiers of plates. With shearing, there are often slight irregularities, and consequently a danger of this occurring. Planing also has the further advantage of removing incipient cracks, indentations, etc., from which fracture may commence. For first-class work, all edges and ends of plates should be planed.

The Web.—The web thicknesses used for ordinary girderwork vary by sixteenths from $\frac{3}{8}$ " to $\frac{7}{8}$ ". Although theoretical requirements would be often satisfied by webs thinner than $\frac{3}{8}$ " thick, practical considerations fix this thickness as a minimum one. This minimum is fixed principally as being the least thickness permissible to provide against corrosion, but at the same time, as far as the actual manufacture is concerned, a more important consideration is that plates thinner than $\frac{3}{8}$ " are very awkward to handle, especially if deep and long, as web plates usually are. Both in deep girders and in shallow girders carrying heavy loads, it will often be found that the web thickness will require increasing beyond that calculated for direct shear, in the former case on account of the greater tendency to buckling and twisting, and in the latter because of deficient bearing area in the riveting.

In the previous chapter it is shown that theoretically the web can be gradually reduced in thickness from the ends to the centre of the girder, i.e. reduced in proportion to the shearing forces, but this is one of the many points in designing, in which theory is overruled by the equally important art of practical design.

In girders of moderate span the web plate will generally be in one length, and, therefore, of the same thickness throughout, but in large and heavy spans, the question of reducing the web thickness towards the centre, is one that needs due consideration. Designs have been made, and are still occasionally seen, in which the web thickness is reduced $\frac{1}{8}$ " at every stiffener, so that there are as many as seven different thicknesses of plates. This is not only cheese-paring, but bad designing. From a practical standpoint, the fewer changes of web thickness there are the better. Every change of thickness means, of course, a joint and covers, and joints mean more work and expense.

The effect on the weight of a girder by changing the web thickness can be easily calculated, and the result will show that the reduction in dead load is practically of no account. On the other hand, the cost of the additional material will in most cases be much less than the cost of workmanship on the web covers, and thin packings which have to be provided under the web covers, and on the springing of the main angles at the joint. Also, when all

the plates in a job are, as far as possible, of one thickness, quicker delivery and identification of the material is obtained. For girders less than 40 ft. long, there is usually no reason why the web plate should not be the same thickness throughout, and for girders up to about 60 ft. long, three thicknesses of plate will be found most economical. There is no necessity for having more than five thicknesses of web in any plate girder, and this number only in very heavy girders.

The sizes of web plates should always receive attention, as steel makers charge extra for long and wide plates, and it may be cheaper to joint the webs frequently than pay increased prices. Another point not to be overlooked is that long, deep, and heavy plates are extremely awkward to handle in the girder yard.

The limiting sizes and areas to which plates are rolled, together with the extras charged for lengths, thicknesses, etc., are given in the Appendix.

Riveting.—The arrangement of the rivets is one of the most intricate points in the detailed design of bridgework, and frequently receives little consideration from some designers, who dismiss it with the statement, “ $\frac{3}{4}$ ” rivets, 4 ins. pitch throughout,” or such like, at the foot of the drawings. There is no need to go to the other extreme and show in every rivet and dimension all broken pitches. If a general statement as to riveting is put on the drawing, all rivets not conforming to this should be shown, and no mistakes can then take place.

From the theory of plate girders, rivets at the ends of a girder must be either of larger diameter or of a smaller pitch than those towards the middle. The most economical method is to keep both the diameter and pitch constant, and in small girders up to, say, 35 ft. long, this can usually be done, but in the case of long and heavy girders one of the alternatives mentioned above must be adopted. The most common and economical method is to keep the diameter of the rivets the same, and vary the pitch. This is obviously the most economical method, as far as the manufacture is concerned; if the diameter is changed, then drills and riveting snaps must also be changed, causing extra workmanship, and loss of time which is avoided if the pitch only is changed. However, in some cases, especially in shallow girders carrying heavy loads, an increased diameter of rivets at the ends is essential. The rivets in the flanges should as far as possible be equally spaced. They are generally spaced to suit those connecting the main angles to the web, although this entails a certain amount of waste towards the centre of the girder, where fewer rivets per foot run would suffice to transmit the increment of flange stress.

It is common practice in America to increase the pitch towards the centre

of the girder, but British practice favours the equal pitch, which is more economical in the long run.

The most common diameters of rivets used in bridgework are $\frac{7}{8}$ ", and 1", and these, being standard sizes, should be used wherever possible, except in light work, such as floorplates and parapets where $\frac{3}{4}$ " diameter rivets are ample.

To avoid confusion, the proper size of the rivets intended to be used should be made clear on the drawings. For instance, a $\frac{7}{8}$ " diameter rivet may mean a $\frac{7}{8}$ " diameter rivet with a larger hole to allow for the expansion of the rivet when heated, the rivet being more than $\frac{7}{8}$ " diameter when closed, and, therefore, stronger than the calculated value, the sectional area of the plates at the same time being reduced, or, on the other hand, if the hole is $\frac{7}{8}$ " diameter the rivet must be less, but the diameter of the rivet when closed will agree with the calculated value. There appears to be no standard rule as to which method to adopt, but it would be better both for designers and manufacturers if, say, the nominal diameter of the rivet represented the diameter of the hole, i.e. a $\frac{7}{8}$ " diameter rivet means a $\frac{23}{32}$ " diameter rivet with a $\frac{7}{8}$ " diameter hole.

When 5" by 5", or 6" by 6" main angles are used, the rivets should always be double riveted arranged in a zigzag fashion, as single riveting is not satisfactory in these cases. Also, when there are two or more flange plates, the rivets through these should be arranged zigzag with those through the main angles and flange.

The calculations for riveting should never be taken too finely, as the rivets are often stressed in many ways other than those usually considered. It is always advisable to increase the calculated number of field rivets in a joint to cover any that may be loose or defective; and an additional 50 per cent. is usually specified.

Care should be taken to see that ample room is left for the proper snapping of rivets, otherwise one head has to be flattened or chipped.

From a maintenance point of view, perhaps the most vulnerable parts of a bridge are the rail-bearers and cross-girder connections, as the effects of impact are always most severe on these parts, and in many old bridges they have proved to be the weakest points. These connections should be very carefully designed, and ample riveting, more than theoretically necessary, should be provided. The riveting is dealt with by hand at site, and oftentimes the girder depth is restricted at the ends, and the position of the rivets cramped, rendering the act of riveting very difficult, and this, coupled with the fact of their being field rivets, never so reliable as shop rivets, accounts for the loosening of rivets so often experienced.

The rivets in the flooring of a bridge should not be more than 3" pitch, if tight work to exclude moisture is expected.

Joints.—The jointing of large plate girders is a question of considerable importance, and one that should receive the closest attention.

The detailed design of joints considered in the previous chapter is almost entirely a theoretical matter, but the position of joints in a girder is a practical consideration, depending upon the length of the girder and upon the methods of transportation and erection.

In detailing the jointing of a girder the designer must first of all arrange the joints so that the lengths of the various plates and sections are not only within the ordinary rolling limits, but also convenient for handling in the yard. When a girder is so long that it cannot be transported wholly in one piece, the position of the joints is then also dependent upon the methods of transport and erection, and particulars of these must be known before details can be arranged.

The capacity of the unloading and erecting cranes has to be taken into account, and if there is to be railway transport the sizes of the girder pieces are of course limited. With spans of 30 ft. and under, no joint whatever is required, but at 40 ft. at least one web joint must be made and, if the girder has to be shipped abroad, a flange joint also, the girder then being in two parts which are riveted at site.

Girders which have only to be transported by railway, can, if adequate erecting tackle be available, be riveted complete in the shops before despatch. The limits of size for railway transport are about 90 ft. in length and 8 ft. in depth.

There is a distinct difference in the arrangement of joints in girders which can be riveted complete, and those which have to be broken into convenient pieces for shipment.

In the first case the joints have only to be arranged to suit the material sizes, and can therefore be placed in the positions best suited to theoretical requirements, that is, with the web joints near the position of least shear, and the flange joints breaking joint with those of the web. When a shipping joint has to be made, both the flange and web must be broken at the same point so that there are no projecting pieces liable to damage during transit.

As before mentioned, joints are not required in girders under about 35 ft. over all. As a general rule, a 40 ft. girder will have one joint, a 50 ft. girder two joints, and girders from 60 to 80 ft. three or four joints. This applies to either girders riveted complete or broken for shipment, the only difference between the two being in the relative positions of the joints in the girder. The maximum lengths which should be used for flange plates without joint

is 35 ft., but in most cases 20 to 25 ft. will be found the most economical length.

Some American Specifications often demand that flanges less than 70 ft. long shall not be spliced, but such a length without a joint is seldom used in British practice.

It is quite easy to obtain angle bars up to 50 ft. long, but bars of this length are extremely awkward to handle without their twisting, so they should not be used except in special cases.

Angle covers are sometimes specified to be made from other angles of smaller size, the square corner being rounded off so as to fit into the rounded corner of the angle covered, as in Fig. 80, *a*. This method should always be avoided, as the covers cannot properly fit against the tapered sides of the angle. Covers for angle bars should be specified to be made either of bent plates, or of larger angles planed down and properly fitted as shown in Fig. 80, *b*.

In girders that can be riveted complete in the yard, both flange and angle



FIG. 80.

joints should be "staggered." For instance, in a girder, say 50 ft. long, requiring one joint in each main angle and flange, the joints should not be opposite each other, but at equal distances on each side of the centre line of the girder. The joints in the near and far main angles of the top flange should correspond in position, and break joint with those in the far and near main angles of the bottom flange, as shown in Examples, Plates Nos. I. and II. It should be noted that each angle is covered separately.

There should never be a joint without a cover, and in all cover plates the fibre of the metal should run in the direction of the length of the member joined, and to ensure this, all covers should be specified to be ordered in long lengths.

It is the best practice for all plate and angle-bar covers to be double, *i.e.* one cover on each side of the joint, and their combined thickness 50 per cent. greater than that of the thickest plate or angle bar covered, while for field joints, the aggregate rivet area in shear should be 50 per cent. in excess of the nett sectional area of the member united.

Spacing of Web Stiffeners.—The proper spacing of web stiffeners is a question to which a considerable amount of attention has been given, and there is, perhaps, no other part of bridge design on which so much has been written or so many theories advanced.

In the practical design of plate girders, little reliance is placed in most of these theories—useful as they are—as the spacing of the web stiffeners in most cases depends on practical considerations, *e.g.* pitch of rivets, position of cross-girders, or pitch of troughs, bracing, etc.

The spacing of web stiffeners, and the web thickness are theoretically interdependent; therefore, in the case of Deck bridges, of square span,

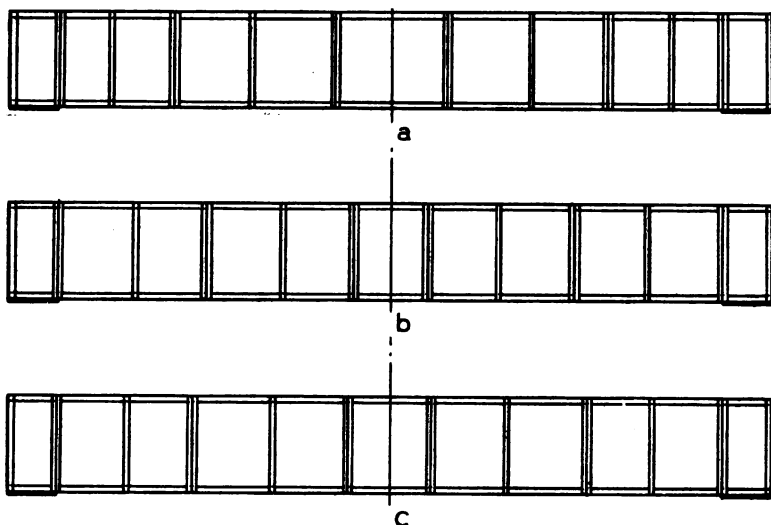


FIG. 81.

where no cross-girder or other connections occur, the stiffeners could be spaced according to theory, *i.e.* the spacing gradually increased as the web thickness decreased, as in Fig. 81, *a*. The best practical method would be to space the stiffeners equally as in Fig. 81, *b*, but the most economical method is obtained by a compromise between theory and practice, and spacing the stiffeners as in Fig. 81, *c*.

There should always be a stiffener over the edge of a bearing plate, and one at every cross-girder, or at points where any concentrated loads occur. Deep girders have, of course, a greater tendency to buckle than shallow girders, consequently the stiffeners in the latter can be spaced further apart than in the former. In girders over about 4 ft. deep, the distance between

centres of stiffeners should not exceed the depth of the girder, with a maximum of 4 to 6 ft.

There are three kinds of stiffeners used : the single-angle stiffener, the tee stiffener, and the plate and angle, or gusset stiffener (Fig. 82).

Single angle and tee stiffeners are used to assist the web to withstand buckling, the latter being the most commonly used, but they have the disadvantage of upsetting the regular pitching of the rivets.

There is no reason why single-angle stiffeners should not be used in preference to tee stiffeners, as they are quite strong enough for the purpose required and do not upset the riveting, while at the same time they are a cheaper material than tees.

Gusset stiffeners are generally provided at points where concentrated loads occur, and over the edge of bearing plates, and in deep and heavy

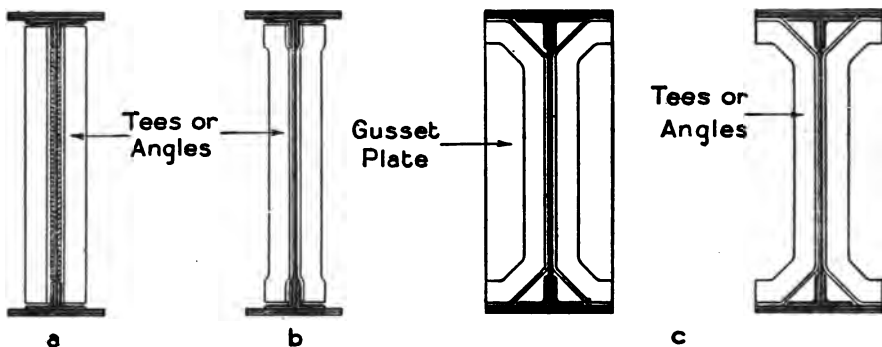


FIG. 82

girders spaced alternately with tee, or single-angle stiffeners. This type of stiffener not only supports the girder laterally, but also assists in preventing any twisting action of the flanges.

The ends of stiffeners are of three types : straight, joggled and kneed (Fig. 82, a, b, c). Straight stiffeners are only used in girders with narrow flanges ; they are packed up to the level of the main angles, as in Fig. 82, a. With joggled stiffeners, the packing is done away with, the ends being joggled over the main angles, as in Fig. 82, b. Kneaded stiffeners (Fig. 82, c) are the type most commonly used, but they can only be adopted when the flange plate projects 3" or more past the edge of the main angle, so as to provide room for riveting. They are more expensive than straight or joggled stiffeners, owing to the extra workmanship required in bending, but they are more effective in preventing any twisting action of the flanges.

Square knee stiffeners (Fig. 83) are sometimes used, but these should

always be avoided as much as possible, as they have to be cut and welded by hand, and there is always a liability to fracture at the crank, especially in the case of tee stiffeners. In addition, square knee stiffeners are very expensive.

Stiffeners placed over cross-girders should always be stopped $\frac{1}{2}$ " short of the top of the cross-girders, so that during erection the cross-girders can be easily slid into position, after which a packing is inserted, and the stiffener riveted to the cross-girder (see Example, Plate I.). This also applies to stiffeners placed over transverse troughing between main girders.

Spacing of Cross-Girders.—The spacing of cross-girders in a bridge depends upon the length of the span, the amount of skew (if any), and the spacing of the axle loads. The principal load on cross-girders is that due to the rolling or live load, which is composed of a series of axle loads. In traversing a bridge each of these axle loads must at some time be carried directly by each cross-girder, therefore each one must be strong enough to carry the heaviest axle load, and since the object of spacing the cross-girders close together is to reduce the load on each, it will be clearly seen that however close they are spaced, no advantage will be gained by spacing them closer together than the spacing of the heaviest axles.

Suppose that the heaviest axles in a locomotive are not closer together than, say 8 ft., it is evident that the load on each cross-girder spaced 4 ft. apart will be practically the same as that on each cross-girder spaced double that distance, or 8 ft. apart.

It follows from the above that the minimum distance apart of cross-girders should be such that the live load transmitted by the longitudinal rail-bearers does not exceed the weight of the heaviest axle, and this generally occurs when the cross-girders are spaced just less than the distance apart of the two heaviest axle loads. There are, however, other considerations which render it more economical to space them further apart than the minimum, but no general rules can be laid down. Broadly speaking, the most economical distance apart of cross-girders will, perhaps, be obtained, when the weights of one cross-girder and the rail-bearers in one bay are about equal.

For plate girder underbridges up to 100 ft. span, the usual practice is to space cross-girders from 8 to 10 ft. apart, and under ordinary conditions this will be quite economical.

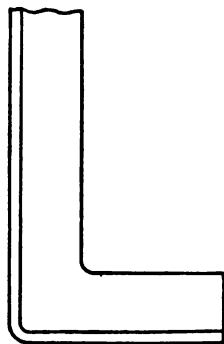


FIG. 83.

Arrangement of Cross-Girders

The arrangement of the cross-girders is one of the first points to be considered in designing a bridge.

In three-girder "through" bridges, cross-girders are often seen with their ends abutting against opposite sides of the centre main girder and secured by the same rivets, as shown in Fig. 84. This practice should always be avoided, it being neither conducive to good workmanship or to an efficient connection, as the rivets in the connections will be subjected to severe treatment as the cross-girders deflect under the load, first on one side and

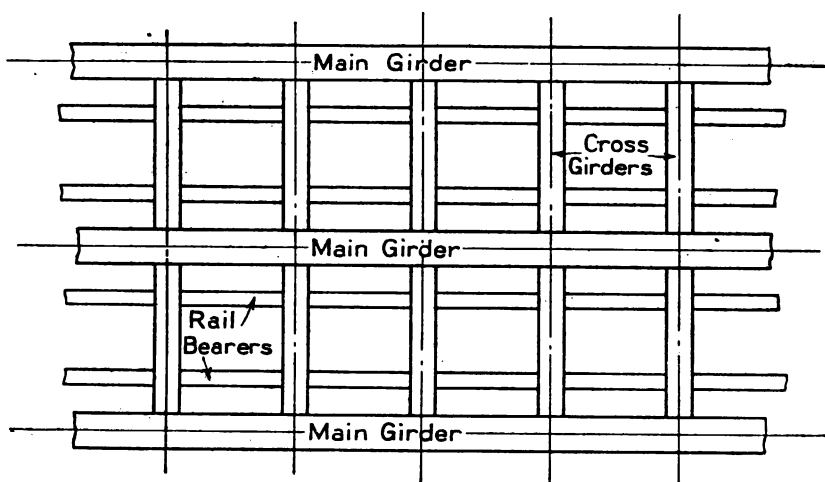


FIG. 84.

then on the other. Cross-girders should always be arranged cross-pitched, or "staggered," as shown in Fig. 85.

It is very desirable in arranging cross-girders to keep the relative position in each main girder the same, and also to avoid having a cross-girder over the bearing and at the centre of the main girder, which latter position would tend to give an increased bending moment. No difficulty will be experienced in arranging the cross-girders in square spans, but skew spans often prove very troublesome, although by careful arrangement it is possible to keep the position of the cross-girders the same in each main girder (except, of course, that the positions are reversed), and also avoid one at the centre, or at the bearing of the main girders.

Parapets.—In accordance with the requirements of the Board of Trade which apply to Railways in the United Kingdom, all bridges under the

railway should be provided with handrails or parapets, not less than 4 ft. 6 ins. above rail level, and with projecting platforms for the escape and protection of platelayers, and in the case of bridges carrying roads over the railway "there must be a parapet on each side of the bridge not less than 4 ft. high."

In towns, bridges carrying the railway over roads must be provided with a close parapet or screen, so that horses on the road beneath cannot get sight of the trains, and when stations occur on or near an underbridge a parapet on each side should be provided, sufficient to prevent passengers falling from the bridge in the dark. There is always a possibility of trains stopping on a bridge near a station, and it is easy to conceive that passengers, thinking

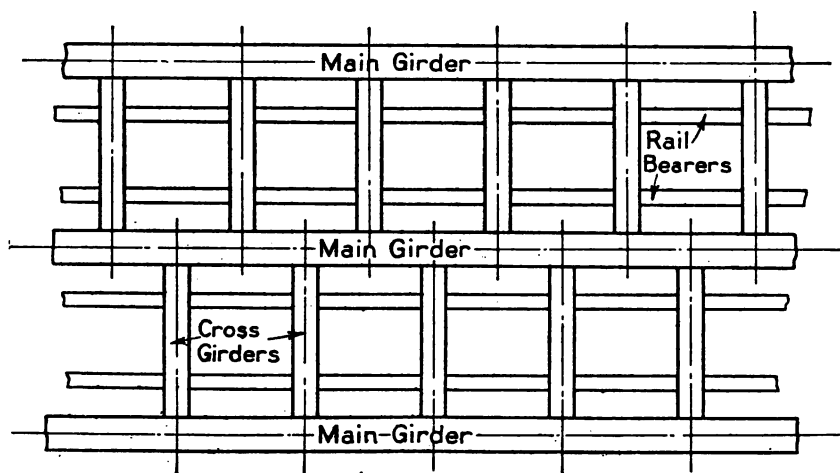


FIG. 85.

the train was in the station, may alight; therefore, a bridge so situated should be provided with parapets sufficiently high to prevent people falling over.

Plate girders are sometimes high enough above the floor or rail level to form their own parapet, as in the case of overbridges of the "through" type.

In the majority of underbridges, however, the outer girders are too near the rails and too low to act as parapets, so special provision must be made for these. The top flange of the girder usually acts as a walking platform for workmen, and if the structure gauge admits, the parapet or handrail can be fixed at the outer edge of the top flange; if not so arranged a wider platform must be provided, and the parapet or handrail fixed as shown in Example, Plate I.

In underbridges over rivers, canals, cattle creeks, etc., open parapets

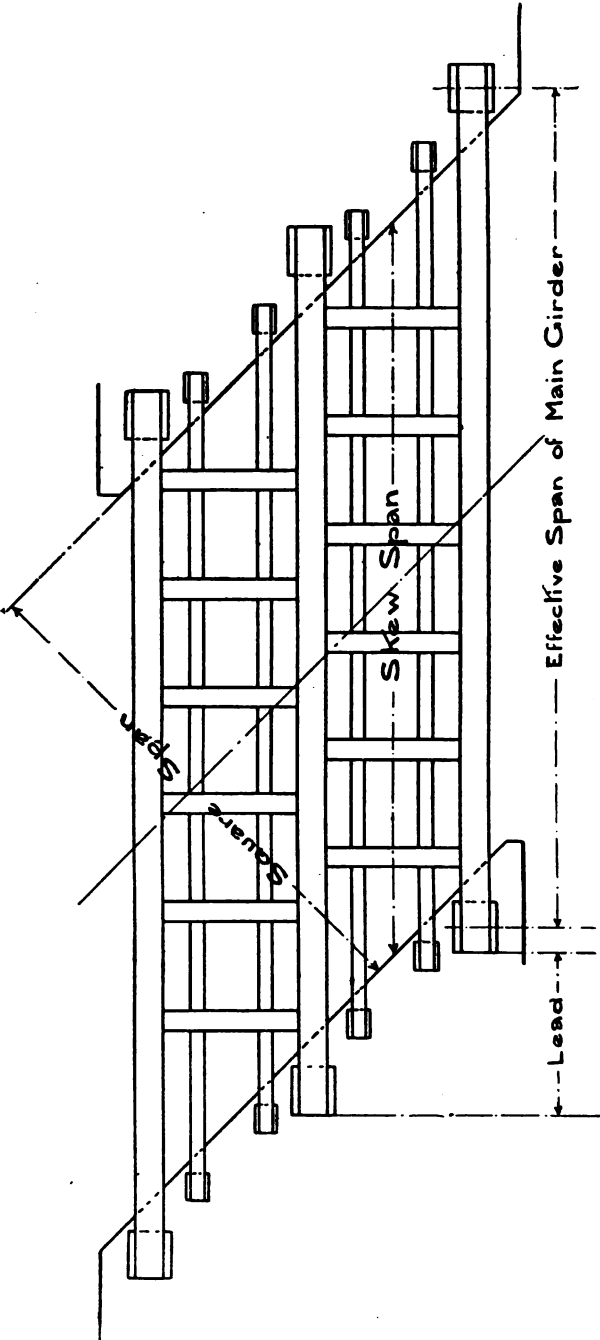


FIG. 86.

are quite sufficient, and these may be composed of angle or tee standards with gas tubing.

Road bridges are generally provided with brick or masonry parapets built on the top flanges of the outer girders. A bed of cement is laid on the flanges to cover the rivet heads, and the parapets built on this.

Skew Bridges.—Bridges very often cross the obstacles to be spanned on the "skew," that is to say, the centre line of the bridge is not at right angles to the opening crossed, consequently the main girders are not directly opposite each other (see Fig. 86).

The skew span of a bridge is obtained by multiplying the square span by the cosecant of the angle of skew, and the amount of "lead" or the distance that one girder is in advance of the other, is obtained by multiplying the distance apart of the centres of girders by the cotangent of the angle of skew.

In designing the main girders of skew bridges there are several important points to be watched.

Tables of uniformly-distributed live loads should not be applied to skew bridges, where there is a considerable angle of obliquity, for the reasons explained in Chapter V., page 55.

The combined effects of the main girder camber and the skew of the bridge should not be overlooked in cases where the angle of skew is very large, else bad cross-girders and other connections will result (see page 94).

Camber.—Main girders are usually built with a camber or upward deflection in the form of an arc of a circle, and the amount generally specified is about $\frac{1}{800}$ part of the span, or 1 in. in 60 ft., measured at the centre of the girder. The girders are constructed so that the camber extends over the whole length of the top flange, and springs from the front of the bearing plates in the bottom flange, so that the bearings may be horizontal.

Camber is given to girders partly to neutralise the deflection, but more so for the sake of appearance, as a straight horizontal girder with several tiers of flange plates, and having no camber, appears to sag.

It is becoming common practice to specify "no camber" for plate girder bridges, and it cannot be said that the finished structures are any less efficient on this account, as camber does not, of course, give additional strength to girders, whilst its omission avoids practical difficulties in manufacture and erection. With skew spans and small square spans, it is perhaps best to have no camber, but in spans of 60 ft. and over, a small amount not exceeding $\frac{1}{750}$ part of the span should be specified, so as to avoid any appearance of sag when the girders are loaded.

Camber is given to girders during manufacture in the yard, and is

obtained by cutting the web plates with the requisite amount of curvature, and pulling the flange angles and plates to it during the plating up.

For small spans it is not necessary to show the camber in the drawings, it being quite enough to specify the amount, but in spans beyond 80 ft. it is better to do so.

In skew bridges, especially in cases where the angle of skew is very large, an important point to be considered is the combined effect of the main girder, camber and the skew of the bridge, which results in throwing the floor system

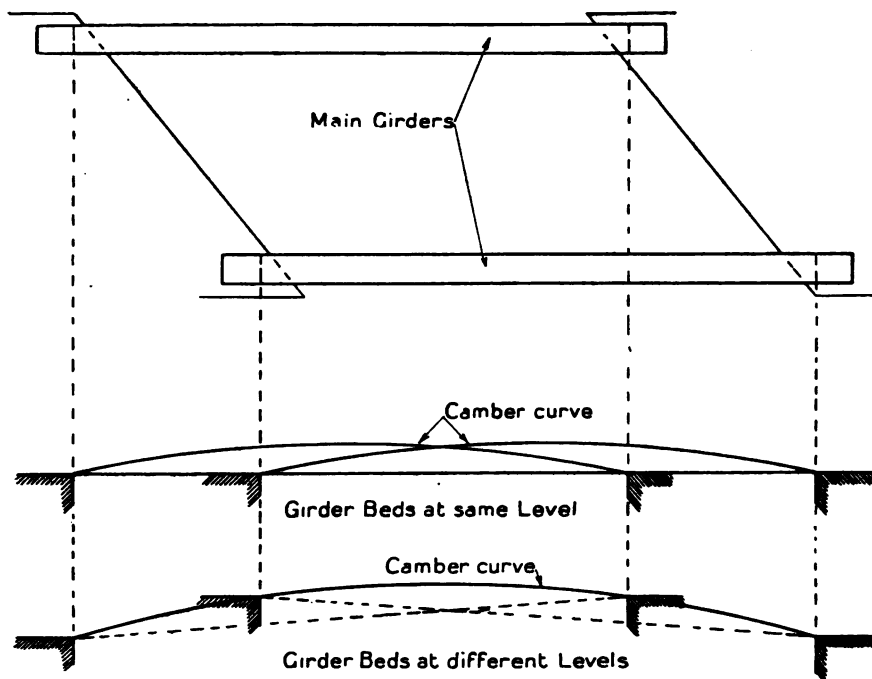


FIG. 87.

out of line, and unless special provision is made for this, bad cross-girder and other connections will be encountered. In the case of a "deck" bridge with girders or troughing resting on top of the main girders, the difficulty can be overcome by using special packings under the girders or troughing to bring all true. In a "through" bridge with cross-girders and rail-bearers it becomes necessary either to set the bedstones of the main girders, etc., at different levels, or to keep them all at the same level and use cast-iron bedplates or packings of different thicknesses, so as to bring the floor system horizontal. The diagrams in Fig. 87 will make this point clear. Where the angle of skew is small the effect may be disregarded.

Girder Bearings.—All girders should be provided with a steel bearing plate at the ends, so as to distribute the reactions evenly over the supports. The length of the bearing plate must be fixed at the start, as the effective span and overall length of the girder depends on it. The effective span, *i.e.* centre to centre of girder bearing plates, is equal to the clear span between abutments, plus the length of one bearing plate.

The area which the bearing plate must present to the bedstones is influenced by the maximum reaction of the girder, and by the nature of the material upon which the bearing plate rests. The allowable pressure on girder bedstones should not exceed 12 tons per super foot, with bedstones of Derbyshire grit or similar stone, and 18 tons per super foot if of granite.

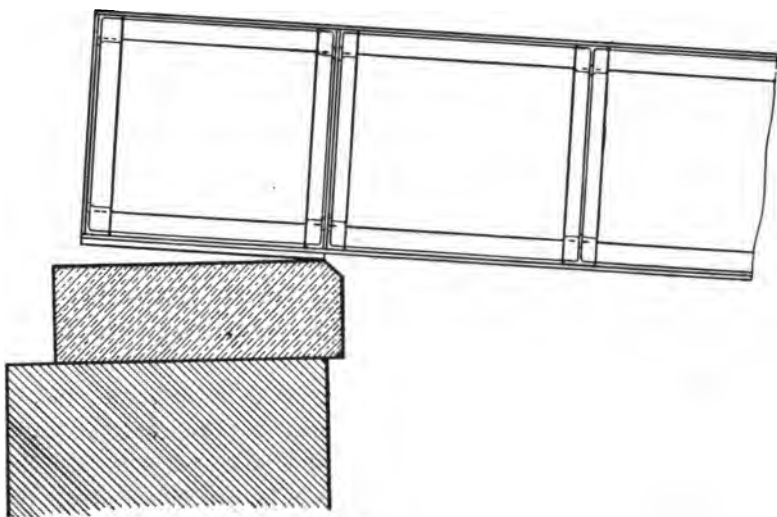


FIG. 88.

The width of the bearing plate should not exceed one and a half times the width of the bottom flange plate, or, with the comparatively thin plate used, the pressure will not be evenly distributed to the bedplate or bedstone.

Long bearing plates should be avoided, as the deflection of the girder has the effect of causing considerable pressure on the inner edge of the bearing plate, and this usually results in cracked bedstones (see Fig. 88).

A good practice is to limit the length of the bearing plate to one and a half times its width.

To allow for the longitudinal movement, or the expansion and contraction of the girders due to variations in temperature, it is necessary to make provision for this at the bearing.

In the case of underbridges, where the girders are over 50 ft. long, special rocker bearings should be provided, but these should be of the simplest type, and not elaborate arrangements as are necessary for large spans. For girders under 50 ft. and over 30 ft. long, cast-iron bedplates should be adopted. These should be of the simplest character possible, but of ample strength. Cast iron is relatively cheap, and the thickness of metal should never be less than $1\frac{1}{2}$ ", for the slightest unevenness in the surface of the bedstones causes them to crack.

Cast-iron bedplates are usually made very shallow, about $1\frac{1}{2}$ " thick, but in the author's opinion, deeper bearings, about 6" deep, as shown in Fig. 120, are much better.

The advantages of deep bearings are, that the underside of the bridge stands well clear of the bedstones, rendering the ends of the girders, etc., accessible for inspection and painting; the height of the masonry can be decreased, and the reactions more evenly distributed, while the bearing area required between the steel bearing plate and the casting is very small.

A layer of hair, felt, or sheet lead, is often placed under cast-iron bedplates to minimise the danger of cracking, or instead of using felt or lead, the bedplate can be wedged up and grouted with cement.

It is rarely necessary to make any special provision for expansion in girders less than 50 ft. long, other than having slotted holes in the bearing plate at one end of the girder, the other end being fixed (see Fig. 115).

Phosphor-bronze bearing strips are sometimes attached to the upper surface of the bedplates at the expansion end of the girder (see Fig. 89). The advantages of phosphor-bronze are that it does not rust and the coefficient of friction of steel on phosphor-bronze is less than that of steel on steel, but it is a very expensive metal and should be used sparingly. The strips, usually 2" to 4" wide, and about $\frac{1}{2}$ " thick, are attached to the upper surface of the bedplate by means of countersunk screws. The spaces between the strips can be filled with tallow for lubrication purposes.

The method of letting the strips into the casting, as is sometimes done, should be avoided as being wasteful and unnecessary.

For girders under 30 ft. long, cast-iron bedplates are not required, the bearing plates resting directly upon the bedstones.

Both the upper and lower surfaces of the cast-iron bedplates should be placed truly level, and the undersides of the girder bearing plates should be made correspondingly level so as to give a perfect bearing.

Girders should be secured to the bedplates by means of tap bolts, the holes through the girder flanges being slotted at one end of each girder to

allow of contraction and expansion ; those in the bedplates to be drilled and tapped.

The bedplates should be fastened to the masonry by Lewis or some other

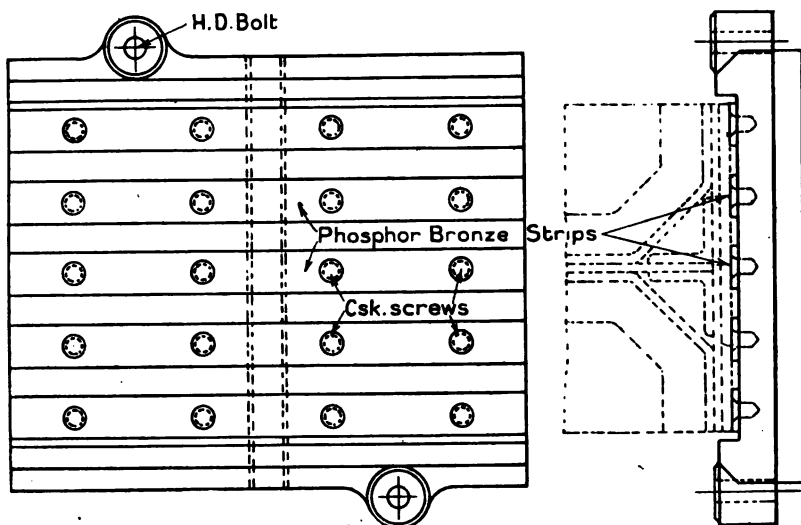


FIG. 89.

form of bolts, so as to prevent any lateral movement of the superstructure. The holding-down bolts are generally from $\frac{3}{4}$ " to $1\frac{1}{4}$ " diameter, depending upon the length of span, depth of girder, etc.

The forms of holding-down bolts generally used are shown in Fig. 90.

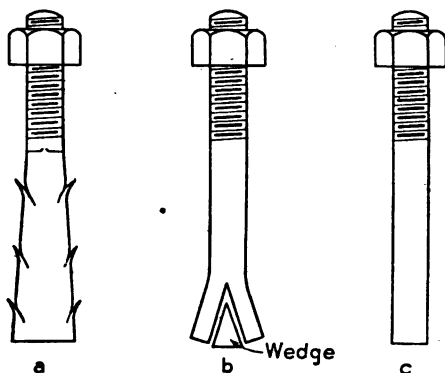


FIG. 90.

The ordinary Lewis bolt (Fig. 90, a), is the type most commonly used. Fig. 90, b shows a wedge bolt, which makes a good holding-down bolt ; the bolt is split at the bottom, and a small wedge inserted, so that when the bolt is

placed in the hole and hammered the wedge expands the lower end of the bolt and prevents its being drawn out. Fig. 90, *c* shows a plain bolt, which is simply placed into the hole in the bedstone and grouted with cement, the

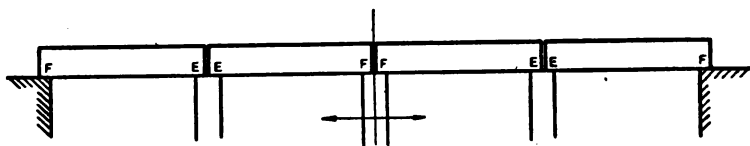


FIG. 91.

force of adhesion between the cement and the steel being considered sufficient to hold the bolt secure.

In cases of a bridge being on an incline, as an overbridge sometimes is, the slotted holes should be placed at the top or higher end of the span.

In the case of a viaduct of several spans on columns or piers, the fixed and expansion ends should be as shown in Fig. 91, the overturning moments on the piers caused by the expansion are then balanced as nearly as possible.

CHAPTER VIII

WATERPROOFING AND DRAINAGE OF BRIDGES

THE draining of bridge floors of all types is a difficult problem, and one requiring careful consideration. A very common defect of many old bridge floors, and one that causes much trouble to Engineers responsible for their maintenance is their lack of watertightness. Water or moisture is the great enemy of all metallic structures, and it is imperative to have a thoroughly watertight floor, and every facility should be given for the flooring to be properly drained, not only to prevent corrosion of the steelwork, which, of course, is of the greatest importance, but also in the case of bridges crossing public highways, to prevent water percolating through to the roadway below, as such bridges, whether crossing main streets in a town or country roads, must be kept so dry that no dirty water can drip through and become a nuisance to people passing underneath. As previously stated, the principal object of drainage is to keep water from contact with the steelwork of the bridge, and so prevent corrosion; if this be efficiently done it is then, obviously, quite a simple matter to keep the water from dropping on to the roadway. Many methods and arrangements are adopted in the endeavour to keep water from the steelwork of bridges, and to provide efficient drainage, and the following examples show very usual methods of overcoming the difficulty.

DRAINAGE AND WATERPROOFING OF RAILWAY UNDERBRIDGES

The transverse section of a "through" bridge with rail-bearer and cross-girder floor shown on Plate I., illustrates the precautions usually taken to provide a watertight floor in a bridge of this type. In each side bay of the steel floor plates two holes $1\frac{1}{2}$ ins. are drilled, over which are fixed cast-iron perforated gratings or grids, with a short piece of galvanised tubing attached, which passes through the hole and discharges the water into half-round or square-shaped gutters, running parallel to the parapets and fastened by straps to the bottom flange of the cross-girders. These gutters in turn discharge through outlets into larger ones running along the abutment faces to a rain-water head connected with pipes running down the face of the abutments in chases cut in the brickwork.

Protection of Flooring.—The flooring carrying the ballast is protected from moisture by a covering or layer of natural rock asphalt, not less than 1 in. thick, the upper surface of the coating falling in all directions towards the drainage grids. It will be noticed that the floor plates over the centre bays are riveted on to the tops of the cross-girder and rail-bearer flanges, while those over the two side bays are riveted to the underside of the flanges. This arrangement has the effect of allowing a good fall to be given to the protective coating enabling the water to be quickly carried away to the drainage grids. To avoid damage to the asphalt from the crow-bars or picks of platelayers, a layer of old bricks, old sleepers, or, where construction depth is limited, a layer of cement should be laid over the asphalt. The best protection is, perhaps, a layer of old bricks with open joints run over with boiling pitch and asphalt.

Carrying away of Water.—In order to deal with the water that runs off the ends of the bridge, and also to allow for painting the ends of rail-

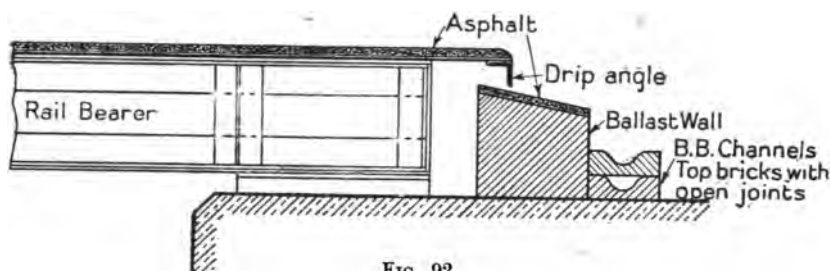


FIG. 92.

bearers or cross-girders that rest on the abutments, the floor plates should be carried well past the ends of the rail-bearers or cross-girders, and finish with a drip angle riveted to the underside, projecting over dwarf walls of brickwork or concrete built with sloping tops, and covered with a layer of asphalt as shown in Fig. 92. The water is carried over the walls into concrete or blue brick channels, and led away to the sides of the embankments or to down pipes. The dwarf walls also serve to keep the ballast away from the bedstones and the steelwork. It is necessary in all cases to provide ballast walls at both ends of the bridge, but if the bridge is built on a gradient it is only necessary to provide for drainage at the low end. Channels to carry away the water are not always provided at the ends of the bridges, it being sometimes considered sufficient to allow the water to run over the ballast walls and down the backs of the abutments. When this is done and the bridge is in cutting, the backs of the abutments are usually built with chases in the brickwork about 14 ins. by 9 ins., and 9 ft. apart, filled with

dry rubble, down which the water percolates into 3 in. diameter pipes built through the abutments. When a bridge is in a bank the chases are omitted, the backs of the abutments being lined with dry rubble and the water carried away as before. In bridges over public roads the water draining away must be discharged into a drain and connected up to the public surface-water sewer, or, if permissible, discharged on to the road.

Protection of Web Plates.—The webs of the main girders must also be protected from contact with the ballast, and in deep girders this is best done by providing thin vertical ballast plates, carried up to rail level, running along the inner sides of the girders, and secured by cleats to the gusset plates or stiffeners (see Example, Plate I.). The floor plates at the sides are only carried as far as the inner edges of the main girder flanges, the open space between the girder web and the ballast plate being covered by a thin removable steel weather plate attached by cleats and bolts to the gusset plates. This arrangement not only keeps the ballast clear of the main girder webs, but also permits of easy painting and inspection from above and below.

An alternative arrangement to the above is shown in Fig. 95. The floor plates, similar to the troughing, as shown, are carried to the main girder webs and cement concrete placed alongside the webs to about 3 ins. above the sleeper level, and finished off with a good slope. To prevent the concrete breaking away from the web plates, a small groove should be formed against the web plate, the asphalt being carried over the concrete and finishing in the groove, which prevents any possibility of a crack appearing there.

Drainage of Deck Bridges.—"Deck" bridges with plated floors are usually drained by drilling holes in the floor plates about 5 ft. apart, into which are screwed short lengths of galvanised gas-tubing, the water discharging through the tubes into gutters; in some cases, when bridges are over unimportant land, there is no need to provide gutters, the water being allowed to drip away.

The ends of the main girders should stand clear upon the abutments and ballast walls built about 12 ins. clear of their ends, the floor plates being carried over the walls, as shown in Fig. 93. This arrangement throws the water from the asphalted floor well clear of the girder bedstones, and enables the ends of the girders to be inspected and painted with facility. In many cases girders have been built into the abutments, and the asphalt continued from the floor plates over the top of the ballast walls without any open space. If this is done it can be easily seen that as soon as traffic commences to run the vibration of the girders produces a crack in the asphalt between the ends of the girders and the ballast wall, consequently, water runs down over

the end plates and finds its way to the steelwork with very injurious effects (see Fig. 94). Bridges with jack arch floors present very little trouble to

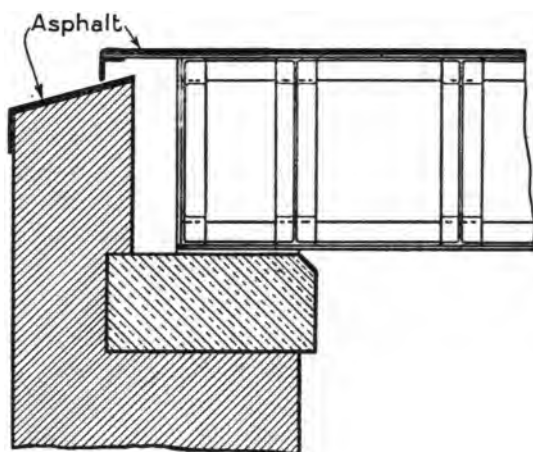


FIG. 93.

drain, as if the arches are built with care and good materials, no water can percolate through to the steelwork. The top of the concrete filling over the

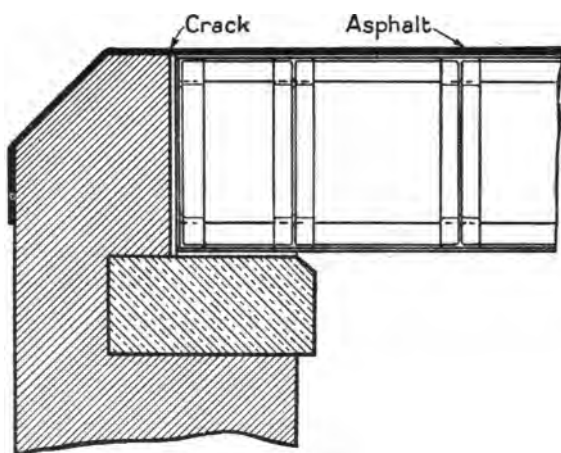


FIG. 94.

arches is given a slight fall from the centre to the abutments and then asphalted, the water being carried down the backs of the abutments.

Drainage of Trough Floors.—Transverse troughing between main girders carrying a ballasted cross-sleeper road, is usually waterproofed and

drained by filling the troughs with bituminous concrete (a mixture of broken stone and boiling pitch) to the level of their upper surfaces, and to some extent above them at the centre of the bridge, forming a summit, and then grading the surface, so as to give a good fall to each end of the bridge. The

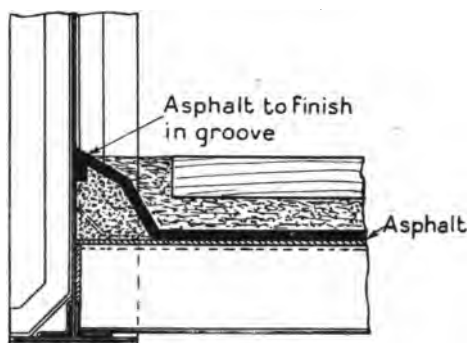


FIG. 95.

concrete is then covered over with a layer of natural rock asphalt 1 in. to 1½ ins. thick, thus forming an impervious surface. The camber of the main girders also helps to give a good fall. It is sometimes the practice to coat the upper surface of the troughing with boiling bitumen, or other preservative,

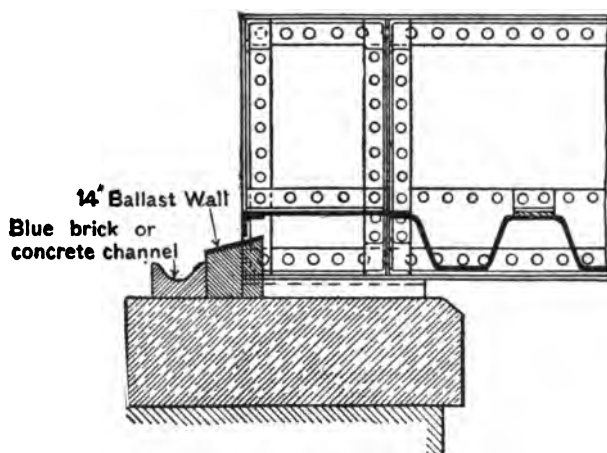


FIG. 96.

before filling in the concrete. The web plates of the girders are protected from contact with the ballast by a filling of concrete, as shown in Fig. 95. The ends of the troughs are finished, as shown in Fig. 96, the water being carried away by brick or concrete channels, or over the backs of the abutments.

When troughing is used with a longitudinal sleeper road without ballast, holes are drilled in the bottom of each trough at the ends, and the water conveyed by gutters, attached to the inner edge of the bottom flange plates,

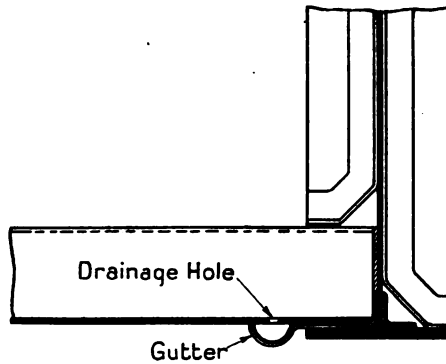


FIG. 97.

the camber of the girders giving the necessary fall towards the down-pipes at the abutments (Fig. 97). An alternative arrangement is to set one main girder slightly higher than the other, sufficient to drain the water to one end

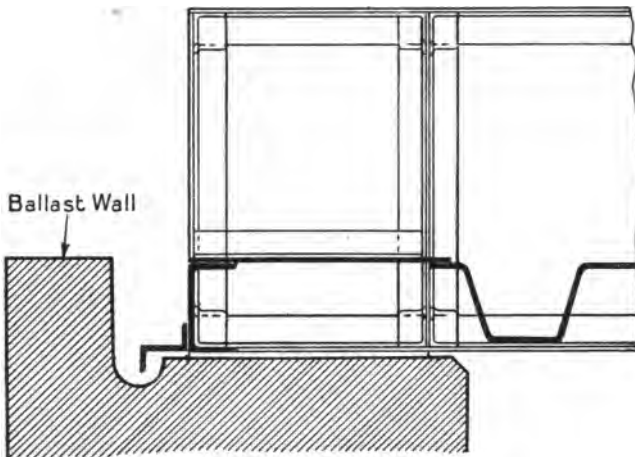


FIG. 98.

of the troughs, the water being carried away by a gutter as before. The ends of the troughing at the abutment can be finished, as shown in Fig. 98.

The top flanges of the main girders are sometimes protected by cement concrete, covered over with a layer of asphalt, held in place between small edging angles.

To enable workmen to walk over the bridge in safety, concrete or brick-

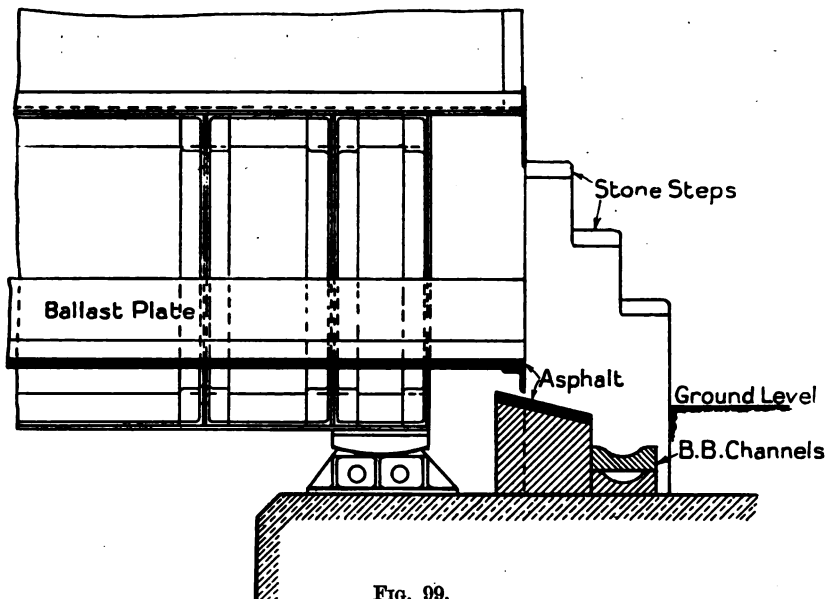


FIG. 99.

work steps are built at the ends of the outer girders, as shown in Fig. 99. It will be noticed that these steps also act as ballast walls, and allow the ends

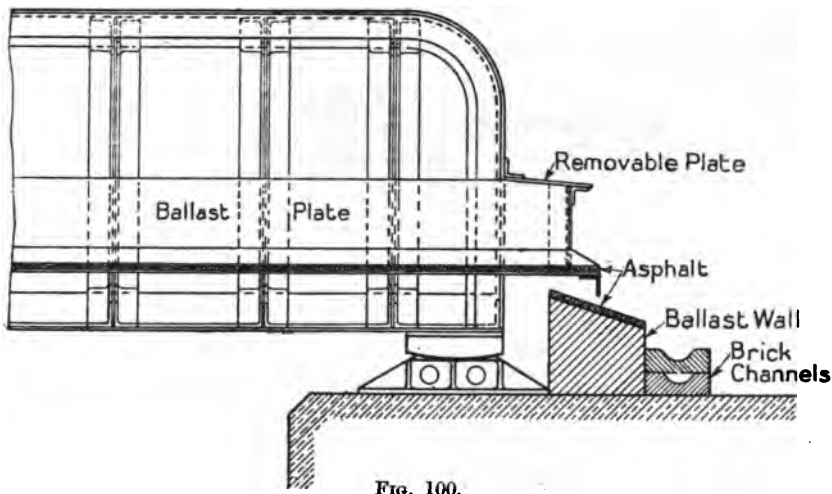


FIG. 100.

of the girders to be thoroughly inspected and painted. The ends of centre girders are finished as shown in Fig. 100. The ballast plates are carried well

past and around the ends of the girder, the open space being covered by a detachable weather-plate as before described.

The provision of removable coverings for painting purposes is, in some respects, a disadvantage, as their removal is often overlooked, but if adopted the special attention of those responsible for the painting should be called to their existence and use.

Drainage of Longitudinal Troughing.—When longitudinal trough-

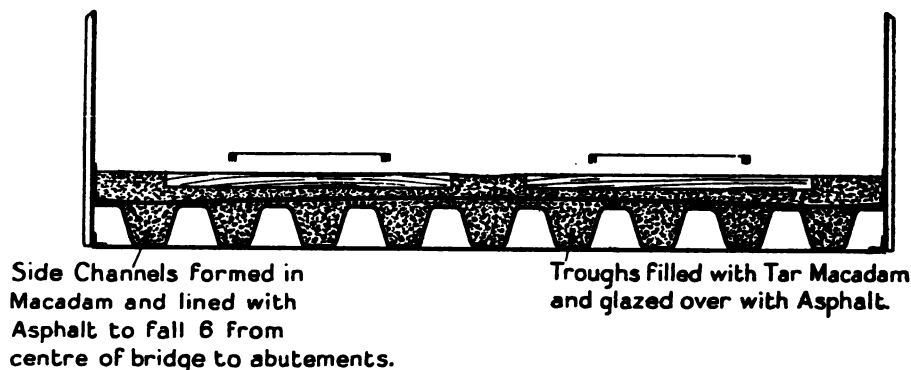


FIG. 101.

ing (Fig. 101) is used for small span bridges, the troughs should be filled with tar macadam to just about their tops, and a good fall given to the macadam from the centre of the bridge to the abutments, and also from the centre to each side. In each outside trough, side channels should be formed in the

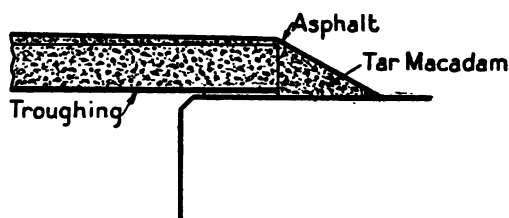


FIG. 102.

macadam, falling 6 ins. from the centre of the bridge to the abutments, and the whole surface of the macadam glazed over with a layer of asphalt 1 in. thick, and carried over the ends of the troughs as shown in Fig. 102. As an additional drainage precaution, nozzles are sometimes screwed into the bottom of each trough, so that if any water finds its way through the filling it can drip away.

Road bridges offer no serious drainage problems; the principal point to

be watched being the waterproofing of the floor, and the main girders where in contact with the road materials. Summarising the above remarks, it is of the utmost importance that all steelwork not permanently covered, or encased in an impervious material, should be open, and readily accessible



FIG. 103.

to inspection and painting, and it should not by any means be in direct contact with timber, ballast or masonry.

The best waterproofing that can be adopted is a preserving layer of cement concrete, covered with a protecting layer of natural rock asphalt.

If care is taken to see that the materials and workmanship are both good, and that a good drainage fall is given to the waterproofing, then there need be no fear of water finding its way to the steelwork.

In the case of bridges abroad, the question of drainage does not usually require any attention, as the rails are generally carried on longitudinal

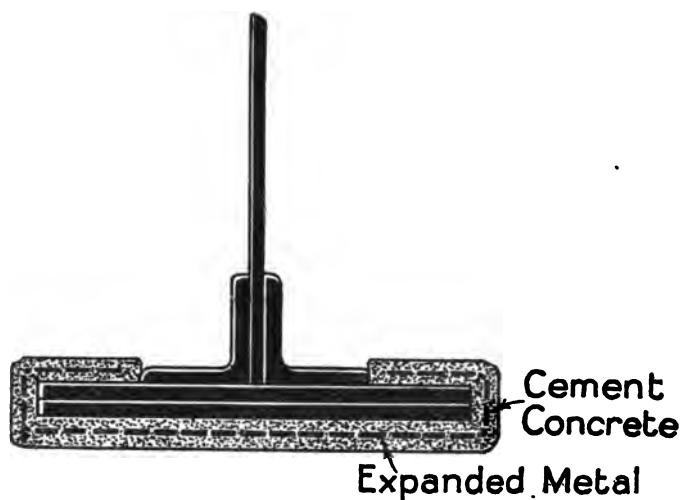


FIG. 104.

sleepers fastened directly to the rail-bearers, no flooring being provided, beyond steel strips from one cross-girder to another for the use of platelayers. Rain water is allowed to drop right through, as in most cases drip does not matter.

Position of Parapets.—To give a more pleasing appearance, it is not uncommon to see both old and new bridges with the ends of the main

girders enclosed in pilasters of brickwork, concrete, or masonry ; but in many cases the pilasters are so built that the ends of the girders are completely encased, rendering any inspection or painting impossible. Pilasters certainly improve the appearance of a bridge, but they should be built a sufficient distance from the steelwork to allow ample room for painting and inspection.

Protection from Smoke.—Bridges over a railway should be provided with smoke plates, as shown in Fig. 103. Instead of using steel smoke plates a good arrangement is to encase the bottom flanges of the main girders in cement concrete with the aid of expanded metal, as shown in Fig. 104.

Rail Joints on Bridges.—Wherever practicable, rail joints upon bridge floors should always be avoided, and this can be done by using rails as long as can be obtained, and crossing the bridge in one length. If rail joints should occur near the centre of a bridge there is a great possibility of a hammering action and working when loads pass over at considerable speed, producing extreme deflection of the bridge, and severe strain in the floor connections.

Ashes and cinder ballast should always be avoided on the floors of steel bridges.

CHAPTER IX

NOTES ON MANUFACTURE

Drawings.—In preparing drawings every care should be taken that these be both accurate and workmanlike. It is essential that all drawings should be accurate, not only as far as the dimensions are concerned, but also as regards details and draughtsmanship.

That this needs mention is evident from the surprisingly large number of designs for simple bridges sent as contract drawings to bridge manufacturers, which have to be redrawn and detailed before they can be sent into the shops as working drawings.

A good drawing should require no revision by the contractor, and be so clear that it can be easily understood by any intelligent plater.

All drawings should be fully dimensioned, so that no scaling is required, and full information given respecting the sizes of the various plates, sections, etc.

All dimensions should be decided with reference to the pitch of rivets, so that joints, length of plates, etc., will be suitable to the pitch of rivets adopted, without having to make alterations during manufacture.

For small spans it is unnecessary to show the camber of the girders in the drawings, so long as the amount is specified, but in long spans of 80 ft. or more, it is better to show it.

The arrangement of the rivets is a point which frequently receives little attention from some designers, who dismiss it with the statement " $\frac{7}{8}$ in. diameter rivets, 4 ins. pitch throughout," or such like at the foot of the drawings. There is no need to go to the other extreme and show in every rivet. The general arrangement of the riveting should be clearly shown, especially where broken pitches or changes in the pitch occurs.

When there are several flange plates, diagrams should be shown, giving the position of the joints, and the lengths, etc., of the various flange plates, main angles and covers (see Examples, Plates I. and II.).

There is no necessity for the elaborate colouring of drawings often seen.

If parts shown in section are coloured, and covers and packings coloured

red and green respectively, both in elevation and section, this is quite sufficient.

Material.—Mild steel is now almost exclusively used for steel bridge-work, and the usual specification is for it to be made by the Siemens-Martin open hearth acid process, or the open hearth basic process.

Formerly, the acid process was very largely used, but at present the latter process is the one most generally adopted by the leading British manufacturers. However, some engineers still prefer the acid process, it being considered the most reliable, but the majority of engineers accept steel made by either process, provided it stands the required tests satisfactorily. The Engineering Standards Committee issue a standard specification for structural steel, and the tests required, although not adopted by all engineers, may be taken as representative of a first-class specification.

The steel is required to be of such a quality as to withstand a tensile stress of not less than 28 tons per sq. in. and not more than 32 tons per sq. in. of section, and the standard test piece must show an elongation of not less than 20 per cent. in a length of 8 ins. before breaking. The material for rivets is required to withstand a tensile stress of not less than 25 tons per sq. in., and not more than 28 tons per sq. in. with an elongation of not less than $22\frac{1}{2}$ per cent., in a length equal to 10 diameters of the piece tested. It must not contain more than 0.06 per cent. of sulphur or of phosphorus. Cold and temper bend tests are also specified for plates and sections, but no bend tests are required for rivet steel.

Template Making.—The first step in the manufacture of bridgework, after the material has been ordered, is the template making. Templates are full-size skeleton reproductions in wood or sheet iron of the different parts of the steelwork comprising a girder or bridge, showing the exact positions of the rivet holes, length of plates, joints, etc.

Thus, for a plate girder, templates would be made for the flange plates, web plates, gusset plates, main angles, etc., and these would be of the same shape and size as the actual material, and show, by actual holes, the exact position of all the rivet holes required in the particular part of the girder represented.

By means of these templates, the positions of the rivet holes, etc., can be accurately marked on the material, so that when the different plates and sections are assembled, all should come true.

Large templates, for web plates and wide flanges, are usually made of narrow strips of $\frac{1}{2}$ in. thick wood or sheet iron, well framed together, while templates for small parts are made in one piece.

Sometimes, in a small job, it is not considered necessary to make special templates for all parts, it being cheaper to set out the rivet holes, etc., in the actual material, and after the holes are drilled to mark off corresponding or similar parts. Unless a large number of similar girders are required, it is not usual to make templates for the web plates of plate girders, the holes being marked off on the actual plate from the main angle and stiffener templates. The shape of the web plate is set out by the plater direct from the drawing and specification.

When wood templates are used, the holes are drilled about $\frac{1}{4}$ in. diameter, to just fit the plater's centre punch ; with well-seasoned wood, the holes can be plugged up and the material used over again.

Straightening.—When the material has been delivered to the girder yard from the steel works, it is first necessary to straighten all plates and bars, as they usually arrive in a more or less buckled or curved state.

Plates are straightened in rolls, or in other words, a bending machine, comprised of three or more rolls. The machines generally used have three rolls, i.e. two bottom rolls, and one top roll, placed above and between the two lower ones.

The plates are straightened by being passed backwards and forwards between the rolls, the requisite pressure being obtained by raising or lowering the upper roll, which is adjustable. Occasionally small, thick plates are straightened by placing them on a surface block and hammering them cold.

Angles, tees, and heavy sections, such as joists and channels, are cold straightened by machinery, but light sections in short lengths are often cold straightened by hand. In the latter method, the bars are placed across a hollow cast-iron block, and struck with a heavy hammer in various places until quite straight.

Shearing and Planing.—The material, after being straightened, is cut to dead lengths and widths. Plates cannot be sheared mathematically exact, so they are generally ordered from the mills with a margin all round the required size. Thus a web plate 20 ft. long by 4 ft. wide nett dimensions, would be ordered 20 ft. $\frac{3}{4}$ in. by 4 ft. $\frac{1}{2}$ in. The usual allowances on plates are as follows :—

Lengths.				Widths.			
Up to 2' 0"	...	...	$\frac{1}{4}$ "	Up to 12"	...	...	$\frac{1}{8}$ "
" 15' 0"	...	...	$\frac{1}{4}$ "	" 12"—20"	...	...	$\frac{1}{4}$ "
" 25' 0"	...	...	$\frac{3}{4}$ "	" 20"—30"	...	...	$\frac{3}{8}$ "
				Over 30"	...	...	$\frac{1}{2}$ "

Special allowances must be made to include camber in ordering web plates (see Fig. 105).

Although planing is almost universally specified, it is often necessary for the manufacturers to shear surplus material from plates, etc., leaving sufficient margin for planing. Shearing is quickly and neatly done by powerful machines, but, of course, it is only possible to shear one plate at a time.

Planing is done by machines, specially adapted for girderwork, and usually several plates are machined on both edges at the same time. Angles, tees, etc., are cut to exact lengths by the cold saw, or by a cropper, an ingenious

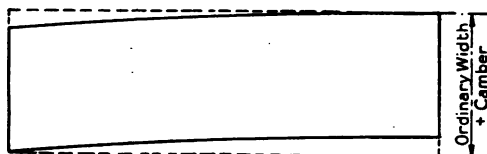


FIG. 105.

machine which crops sectional material cleanly and with as good finish as if cold sawn.

Marking.—After the plates and bars have been planed or cut dead true to the dimensions required, the positions of the rivet holes are marked off from the templates.

The usual method of marking off is by temporarily attaching the template with clamps to the plate or bar it represents, the centres of the rivet holes being then marked through the template on to the material by a centre punch. Afterwards, a piece of tube dipped into white paint is pressed into each hole, making a white ring round the centre punch mark in the material, thus enabling the workmen to see at a glance the positions of the holes, when the template is unclamped.

In some yards, instead of this latter procedure, the plates, etc., before marking, are painted white with a water paint, then when the centres of the holes have been punched through the templates, and the latter unclamped, the diameter of the hole is scribed on the material with a fixed compass. The material is then punched or drilled as required by the specification.

Drilling and Punching.—Practically all Engineers now specify that all rivet and bolt holes must be drilled out of the solid, no punching being allowed. With this method all the plates forming one flange of a plate girder are piled in position and securely clamped, the centres of the rivet holes are then marked on the top plate, and the holes drilled through the whole thickness of flange at once. Other parts of the girder are drilled separately, and after all frazes or burrs have been removed, the different parts are assembled.

When nipple punching, or the "broach and drill" system, is allowed, the holes are punched $\frac{3}{8}$ " to $\frac{1}{4}$ " smaller diameter than the finished hole, and the plates and sections put together in their proper positions, securely bolted up, and the rivet holes drilled or rymered out to the full diameter, through the whole thickness at once.

The holes at field joints, whether drilled or punched, are made to a smaller size at first, then when the girders are built to the exact camber curve, the holes are drilled out to the full size by a portable drill, thus ensuring a perfectly smooth and true hole through all thicknesses.

METHODS OF RIVETING

Hand Riveting.—Although riveting by hand has now been almost entirely superseded by machine riveting, it is still occasionally resorted to, more especially for riveting at site. The rivets are heated by putting them through holes in a plate placed over a coke breeze fire, which enables the shanks to be made red-hot without burning the tails. One rivet at a time is taken from the perforated plate and pushed through the rivet hole, the tail being kept close to the plate by a man, usually called the "holder-up," pressing against it with a "dolly," whilst two other men hammer down the projecting "clink," and the head being made conical by the hammers alone, or finished with the aid of a cup-shaped die or "snap." Hand-made rivets can usually be distinguished by the flatness of the heads, and by the marks on the plates caused by the "snap."

Hydraulic Riveting.—Riveting by hydraulic machinery is used to a very great extent in the girder yard, and is by far the best and most reliable method of riveting. The red-hot rivet is placed in the rivet hole, and pressed between two dies actuated by water-pressure of great intensity, which forms the head and compels the rivet to completely fill the hole.

Hydraulic riveting should always be specified for heavy sections and where several thicknesses are to be joined together. The head of the rivet is very liable to be formed eccentrically in hydraulic riveting, owing to the dies in the machine being out of line with the axis of the rivet. Hydraulic riveting can usually be picked out by a rim extending round the bottom of the head.

Pneumatic Riveting.—This method of riveting is performed by apparatus actuated by compressed air, and has found great favour with manufacturers of constructional steelwork. The appliances used in pneumatic riveting comprise a riveting hammer, and a holder-on, both small, portable hand tools of ingenious construction. Compressed air taken from

a receiver fed by a pump, is conveyed by means of flexible tubing to the riveting tools ; the heated rivet is placed in the hole and the tail held against the plate by the holder-on ; the operator places the riveting hammer against the projecting shank, presses the trigger, which allows the compressed air to act on the piston and form the rivet head by a rapid succession of blows.

Pneumatic riveting should not be employed for heavy work, or when several thick plates have to be riveted together, as, for example, flanges composed of five or six plates.

For general girderwork, and for site connections, pneumatic riveting produces excellent work. Care should be taken to see that the rivet head is not formed eccentrically, as this is very apt to occur with careless workmen.

Remarks on Riveting.—In perfect riveting the shank of the rivet should completely fill the rivet hole, both when it is hot and also when it is cold, but owing to the lateral contraction of the rivet in cooling the latter condition does not actually occur.

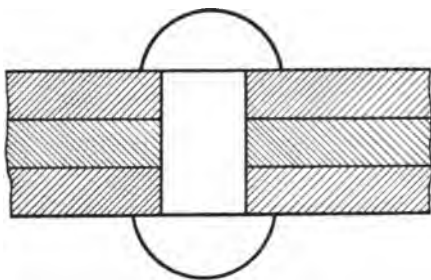


FIG. 106.

The rivet in cooling, also contracts longitudinally, and nips the plates with great power, causing a tension on the rivet ; in using very long rivets, this tension may be so great as to cause fracture of the rivet, so, to avoid this, it is usual to cool the tail end in water before putting it in the rivet hole. The longitudinal contraction also produces frictional

resistance to slipping between the plates, a benefit that designers purposely neglect.

Whatever method of riveting is used, defective work is likely to occur, usually the result of careless workmanship. A common defect in riveted work is shown in Fig. 106, where the rivet head is formed eccentrically, due to carelessness in handling the riveting tools. Such rivets are easily detected, being visibly out of line, and usually have a round under edge on one side. When plates are individually perforated, either by drilling or by punching and rymering, it is quite possible when the plates are put together that the collective hole may be as shown in Fig. 107.

Whether the rivet completely fills the hole or not, non-cylindrical holes are very bad. If the rivet completely fills the hole, the series of sharp angles in the shank greatly assists the shearing stress, while, on the other hand, if the rivet does not completely fill the hole, a full bearing is not obtained, and the rivet is very liable to become loose.

In Fig. 108 two inaccurately perforated plates form a "blind" hole, generally resulting in the rivet partly filling the hole. Very often, in cases such as these last two, the workmen endeavour to bring the holes into agreement by driving into them conical drifts. This is a most objectionable method, as drifting not only smooths the rivet holes, but also injures the plates.

Loose rivets are frequently met with, and can be detected by the sound emitted when tapped with a light hammer. Defective rivets may also be caused by burning the head during heating, as the presence of dirt, or of scale, prevents the metal from properly filling the hole.

All loose rivets, and those with cracked, badly formed, or deficient heads, or with heads that are not truly concentric with the shanks, or of which the

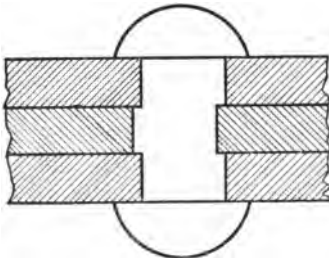


Fig. 107.

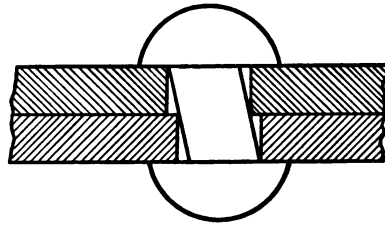


Fig. 108.

undersides of the heads do not bear truly on the surface of the plate or bar, should be cut out of the work and replaced by sound ones.

Care should be taken to see that the riveting snaps do not cut into the material.

During riveting up, all plates, angles, etc., should be tightly held together with bolts, one in every third hole, so as to ensure perfect contact of the surfaces being maintained. This is important, as loose rivets are often the result of the plates, etc., not being in close contact during riveting.

In riveting up girder flanges, the riveting should not proceed first along one side and then along the other, as this method tends to distort the flange plates, but each row of rivets across the whole breadth of the flange should be riveted up as the work proceeds.

When riveting is performed by pneumatic tools, difficulty is sometimes experienced in closing up the plates, etc., so that the surfaces are in proper contact. No difficulty occurs in closing up, two or three plates $\frac{1}{8}$ " or $\frac{5}{8}$ " thick, with $\frac{3}{4}$ " or $\frac{7}{8}$ " dia. rivets, but when four, five, or six plates $\frac{5}{8}$ " thick, with $\frac{7}{8}$ " and 1" dia. rivets, have to be closed, hydraulic riveting is to be preferred. In hydraulic riveting, the shank of the rivet is pressed into the hole and the

head formed by sheer pressure of great intensity, consequently the plates are nipped tightly together and brought into closer contact, and, if thought desirable, the pressure on the rivet head may be sustained till it cools sufficiently to keep up the nip given by the riveting machine.

Inspection.—It is the usual practice for all work to be inspected during manufacture by the Engineer or his representative.

The inspection of work is a very important matter, influencing to a more or less degree both the cost and quality of the workmanship.

On the one hand, if the inspection is likely to be very exacting or unreasonable, the contractor increases his prices accordingly; on the other hand, if the inspection is lax, there is a possibility of the workmanship being of an inferior character.

This latter statement does not imply that contractors take advantage of any lack of inspection, as the name of some firms, who have a reputation to keep up, is quite guarantee enough, in itself, for first-class work.

Good inspection depends upon a reasonable interpretation of the requirements of the specification, both in letter and in spirit, therefore it is essential that an inspector should be practical, understand workmen, and have received a works training.

The work should be inspected at all stages of manufacture, and not be left until all parts are riveted together and painted, as is sometimes done.

The chief points to be observed during inspection are—

That all plates, sections, etc., are of exact and equal dimensions, holding up fully to those marked on the drawings, and are free from blisters, laminations, cracked edges and other defects.

Planing of all edges and ends of plates when so specified.

Drilling from the solid, if punching and rymering is not allowed.

Removal of fraze round rivet holes.

Riveting. Loose and defective rivets. The rivet heads should be tapped with a light hammer, the sound emitted being a guide to the soundness or otherwise.

Correct pitch and size of rivets.

Trueness of abutting ends and edges over their full areas.

Full contact of surfaces over their whole areas, of plates, bars, etc., riveted together.

Lengths of girders, flange plates, and position of joints.

Requisite amount of camber.

Cleanliness of all work before any painting.

Soundness of castings.

When any work submitted for inspection is absolutely rejected on

account of defects of material or workmanship, it should be so marked or defaced as to render it impossible for such to be used in the work.

The defacement should be in the case of plates, by shearing off a corner, and in the case of castings by their being broken.

All loose rivets, or those with badly-formed or deficient heads, should be cut out and replaced by sound ones. The fraze round rivet heads, often seen in hydraulic riveting, is quite permissible, and no objection should be raised to this.

Inspectors should not expect constructional steelwork to have the same degree of finish and accuracy as high-class machinery, but at the same time they should insist upon the work being up to the standard required by the specification.

ERECTION

The erection of bridges is a question of considerable importance, and one that must be borne in mind when deciding upon the type of bridge for any particular situation.

The method of erection depends principally upon the site, the class and type of the bridge, and the exigencies of traffic.

Little difficulty is usually experienced in the erection of new bridges in places where no bridge existed before, as in the case of bridges on a new line of railway, when the steelwork can be erected in position and completely riveted up without any restrictions whatever.

It is quite a different matter in the case of renewals, which generally present many complex and difficult problems for solution. The renewal of railway bridges, involving as it usually does interference with the running lines, is an operation attended with grave risks, and accompanied more or less by delay to traffic, which causes considerable inconvenience and expense. Perhaps in no other class of work is the Engineer required to exercise more resourcefulness than in the renewal and erection of bridges on an important length of railway, where the volume of traffic is such that no risks can be run and no time wasted.

It is scarcely possible to lay down any hard-and-fast rules for the method of erection, as several methods may be employed for the erection of any particular bridge, but the one best suited for the purpose depends upon a full knowledge of all the conditions of the case under consideration.

In the case of new bridges, where there are no traffic restrictions, *e.g.* bridges on a new line, one of the simplest methods of erection is by derrick poles. One or two poles are used, according to the length and weight of the steelwork to be lifted, and these are erected in a convenient position and

suitably guyed. The girders are raised by means of a block and tackle, and when sufficiently high are pulled over the bearings by ropes attached to the ends, and then lowered into place.

Another method of erection frequently adopted in suitable cases, is that of rolling forward the girders from one abutment to the other.

The girder is delivered to one side of the bank, being placed at right angles to the abutments, and round timbers which act as rollers are inserted underneath. The girder is then pulled forward by means of a winch placed on the opposite bank, and when it has travelled far enough the leading end is attached to a jib crane, which keeps it up until the bearing is reached.

In the renewal of railway underbridges the method of erection depends upon the type of bridge to be renewed, the type of the new bridge and the exigencies of traffic. In the case of double-line bridges of the three or two-girder "through" type, a method of erection which finds great favour with Railway Engineers is as follows: Two timber stages are built, one on each side and close to the structure to be renewed, and upon one of these the new superstructure is erected and completely riveted up. During the time this work is in hand the bedstones are renewed and set to the levels required for the new bridge. After this is done, the Engineer is given entire possession of the lines for a few hours, between the running of trains, on some convenient Sunday. The two structures are coupled together and jacked up on small trollies or rollers, running on rails laid from one stage to the other, and then by means of winches, pulled along the rails, until the new structure is in its final position, and the old one over the other stage. The new structure is now jacked up, and after the trollies, etc., have been removed and the bed-plates placed in position, released down on to its bearings.

This method of erection is one that causes the least interference to traffic, as the whole operation of drawing in can be done in a very short time. One of the leading railway companies have erected a bridge by this method in the incredibly short time of $3\frac{1}{2}$ hours. It was a double-line bridge, and the weights of the old and new superstructures were 100 tons and 150 tons respectively. Possession of the line was given at 5.15 a.m. and at 8.45 a.m. a train went over the new bridge. In this instance the rails, etc., had been laid on the new bridge before drawing in commenced.

In some cases, when the time between the trains admits, the old structure can be removed by breakdown cranes working at each end of the bridge, the new superstructure being erected on a temporary stage and pulled in as before.

If this is done, the rivets in the connections of the floor system of the old bridge usually have to be cut out and replaced by bolts beforehand, so that

when the day for dismantling arrives, no time is wasted. This method is only adopted in cases when the old bridge floor has few connections that can be easily cut out. The only saving is in building one stage instead of two, but what is gained in this direction is very small. It is much better and safer to build two stages, and draw the old structure out complete, at the same time as the new one is drawn in, the old structure can then be dismantled at leisure.

In situations where renewal by drawing in is precluded, single-line working has to be employed, one-half of the bridge being dealt with at a time, while all traffic is diverted over the other half, or over loop-lines. When single-line working is adopted, some advantage may be gained by interlacing the two lines, as shown in Fig. 109. This arrangement avoids the use of points and secures a minimum of delay to trains.

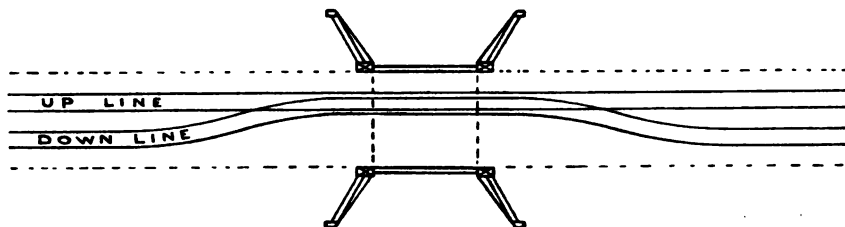


FIG. 109.

The greatest care should be exercised in planning and scheming all erection operations, and nothing should be left to chance. This especially applies to the renewal of existing railway underbridges, where the safety of the line must be assured, and traffic interfered with as little as possible.

It is a good plan to prepare a programme of operations beforehand, allocating a time for carrying out the various operations.

Care should be taken to see that all timber trestles are well made and properly braced. Spare tackle, such as jacks, blocks, winches, chains, etc., should always be provided, so as to avoid any delay to the work in case of breakages.

The question of erection should always be borne in mind when preparing designs, and the work to be done at site should be reduced to a minimum, as this costs considerably more than work done in the yard. The cutting out of rivets, cutting plates or any fitting at site should be avoided as much as possible.

EXAMPLE I

DESIGN FOR AN UNDERBRIDGE WITH TROUGH FLOORING TO
CARRY A SINGLE LINE OF RAILWAY OVER A ROAD**Assumed Data.**

Level of roadway	...	...	...	...	150.00
Rail level over bridge	...	...	...	...	167.58
Minimum headway under bridge	...	...	...	...	15 ft.
Width of roadway	...	...	...	...	34 ft.

Live load. Type locomotive. See diagram, page 56.

Allowable Stresses.*Main Girders.*

Compression	...	...	4 tons per square inch.
Tension...	...	...	5 " "
Web	...	...	3 " "

Rivets.

Shear	...	...	4 tons per square inch.
Bearing	...	...	8 " "

Stress in floor not to exceed 4 tons per square inch.

In calculating the flange sections, nett areas to be taken in tension flange and gross areas in compression flange, the vertical limbs of main angles to be disregarded.

Considering first the design of the floor. The level of the roadway under the bridge is 150.00, and the minimum headway required 15 ft., making the level of the lowest part of soffit of the steelwork 165.00. The rail level over the bridge is 167.58, therefore, the available depth of construction is 2' 7".

The distance from the rail to underside of sleeper is 1' 0½", the depth of ballast 4", and the asphalt 1½", making a total of 1' 6", leaving 1' 1" for the distance from top of trough to lowest part of the steelwork.

A suitable section of trough for this construction is shown in Fig. 110. The depth over the cover is 10¼", the thickness ⅝", and the pitch 2' 0".

For purposes of calculation it is assumed that the troughs are free at the ends, and the live load distributed over two troughs.

The distribution of the load, and the amount to be allowed on each transverse trough is a moot question. The interposition of rail and sleeper between wheel and trough has the effect of distributing a certain amount of the load to the troughs adjacent to the one immediately under the axle load. When the pitch of the troughs is not less than 2' 0", the load should not

be assumed to be distributed over more than two troughs, that is, half the load on one axle on one trough.

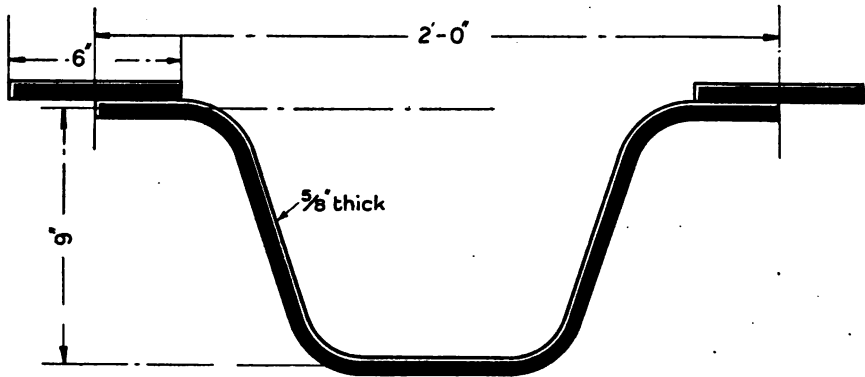


FIG. 110.

From the diagram given on page 56, the heaviest axle that can be brought over one trough is 20 tons, and as this load is really carried by

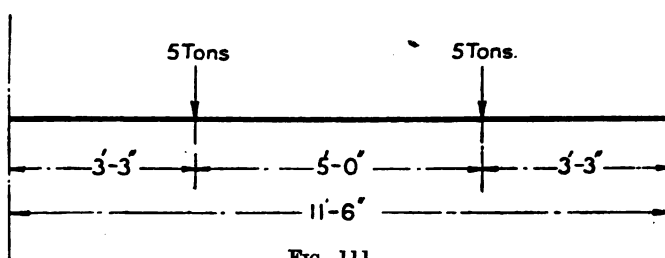


FIG. 111.

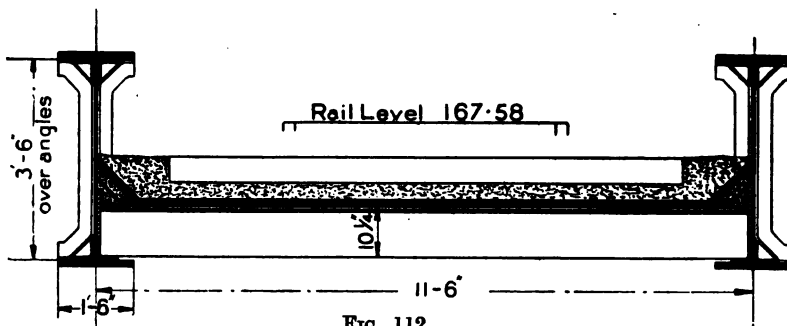


FIG. 112.

two troughs, the load on each trough is 10 tons, or 5 tons under each rail, as shown in Fig. 111.

The main girders will be spaced 11' 6" centres of girders (Fig. 112).

The bending moment under each load due to the live load is $5 \times 3.25 \times 12 = 195$ inch-tons. This is also the bending moment at the centre of trough.

The dead loads on one trough are as follows :—

	Tons.
Trough, $11.5 \times 2 \times 46$ lbs.	0.47
Ballast, $\left\{ \begin{array}{l} 11.5 \times 2 \times .58 \\ 11.5 \times .66 \times .75 \end{array} \right\} = 19.03$ cu. ft. @ 120 lbs.	1.02
Asphalt, $11.5 \times 2 \times .12 = 2.76$ cu. ft. @ 156 lbs. ...	0.19
Permanent way, $2 \times 175 = 350$ lbs.	0.15
	1.83

The bending moment at the centre of the trough due to the dead load
 $= \frac{1.83 \times 11.5 \times 12}{8} = 31.56$ inch-tons.

The total bending moment at the centre of the trough is $195 + 31.56 = 226.56$ inch-tons.

The modulus of section for the section of trough adopted, is 66.3 inch-units.

NOTE.—The properties and section moduli of the various sizes of troughs are given in the handbooks issued by manufacturers of troughing.

It has been shown that the bending moment must be equalled by the moment of resistance. This is represented by the equation—

$$M = fZ$$

where M = Bending moment in inch-tons.

f = Working stress in tons per square inch.

Z = Modulus of section in inch-units.

In the case under consideration—

$$f = \frac{226.56 \text{ (Bending moment)}}{66.3 \text{ (Modulus of section)}} = 3.43 \text{ tons per square inch.}$$

which stress is within the limit permissible.

Design of Main Girders.—The width of the roadway over which the bridge has to cross is 34 ft., so assuming that the bearing plates at each end of the girder are 2' 9" long, and that the front edge of each is 3" from the face of the abutment, the overall length of the girders is 40 ft.

Therefore, the effective span, *i.e.* the distance between centres of bearings, is $40 - 2.75 = 37.25$ ft.

If the effective depth of the girders, *i.e.* the depth over the backs of the main angles, is made 3' 6", this gives a ratio of depth to span of 10.6.

The loads on each main girder will be as follows :—

Dead Load.—

	Tons.
Girders own weight, say	5·0
Ballast, $37·25 \times 5·75 \times 58 = 124·23$ cu. ft. @ 120 lbs.	6·65
Asphalt, $37·25 \times 5·75 \times 12 = 25·7$ cu. ft @ 156 lbs. ...	1·80
Troughing $37·25 \times 5·75 = 214$ sq. ft. @ 47 lbs. per sq. ft.	4·50
Permanent way, $37·25$ ft. @ $\frac{175}{2}$ lbs. per ft. run ...	1·45
	19·40

say 20 tons.

Live Load.—Referring to the diagram on page 56, the equivalent uniformly distributed live load for a span of 37·25 ft. is 2·58 tons per foot run for a single line of railway. Therefore the live load on one main girder is $37·25 \times \frac{2·58}{2} = 48·05$ tons.

The total loads on one main girder = $20 + 48·05 = 68·05$, say 70 tons uniformly distributed.

The maximum bending moment at the centre of the girder

$$= \frac{70 \cdot 0 \text{ (Loads)} \times 37 \cdot 25 \text{ (Effective Span)}}{8} = 326 \text{ ft.-tons.}$$

The maximum flange load at the centre of the girder

$$= \frac{326 \text{ (Bending Moment)}}{3 \cdot 5 \text{ (Effective Depth)}} = 93 \cdot 15 \text{ tons.}$$

Area required in Flanges at the Centre of the Girder.

Top or Compression Flange.—The allowable stress in compression is 4 tons per square inch, therefore the area required in the top flange at the centre = $\frac{93 \cdot 15}{4} = 23 \cdot 29$ square inches.

Bottom or Tension Flange.—The allowable stress in tension is 5 tons per square inch, therefore the nett area required in the bottom flange at the centre = $\frac{93 \cdot 15}{5} = 18 \cdot 63$ square inches.

Composition of Flanges.

Top or Compression Flange.—This flange being in compression, it is assumed that the strength is unimpaired by the rivet holes, so no deduction

is made for these, but in each flange the horizontal limbs only of the main angles is considered in the flange section.

$$\begin{array}{rcl}
 2 \text{ Angles, } 4'' \times 4'' \times \frac{5}{8}'' & = & 5 \text{ square inches.} \\
 2 \text{ Plates, } 16 \times \frac{5}{8} & = & 20 \quad , \\
 \hline
 & & 25 \quad ,
 \end{array}$$

Bottom or Tension Flange.—This flange being in tension it is assumed that the area of the flange available to resist the tensile stresses, is the nett area of the metal, i.e. the area of the plates and horizontal limbs of the angles comprising the flange, minus the area of the rivet holes at the centre section of the girder. With flanges 16" wide, there will be 4 rivets almost on the same centre line across the width of the plate, and as previously explained in such cases the diameters of 3 rivet holes are subtracted from the width of the plates. The diameter of the rivet holes is taken $\frac{1}{8}$ " larger than that of the rivet. The flange will be as follows:—

$$\begin{array}{rcl}
 2 \text{ Angles, } 4'' \times 4'' \times \frac{5}{8}'' \text{ less } 2 \frac{1}{8}'' \text{ dia. holes} & = & 3.83 \text{ sq. ins.} \\
 2 \text{ Plates, } 16'' \times \frac{5}{8}'' & , \quad 6 \quad , & = 16.49 \quad , \\
 \hline
 & & 20.32 \quad ,
 \end{array}$$

It will be noticed that in both flanges the area of the section is a little more than necessary, it being better to have all plates and angles of the same section, than to have odd sections and plates of different widths and thickness, so as to exactly balance the area required by the calculations. In both top and bottom flanges, the main angles, and the first flange plate must for practical reasons extend for the whole length of the girder, but the second plate can be cut short, as the flange stress, being greatest at the centre of the girder, diminishes towards the ends. The simplest way of ascertaining the points at which the flange plates may be curtailed is shown in Fig. 113.

Draw AB equal to the effective span, viz. 37' 3" to any convenient scale, and at the centre erect a perpendicular CD, equal to the maximum bending moment, viz. 326 ft.-tons to any given scale. On AB construct a parabola, ADB, which represents to the scale adopted, the bending moments at every point of the main girder. To find the points where the flange plates may be stopped it is only necessary to set up on the base line AB to the same vertical scale as the diagram, the moments of resistance of the main angles and plates as shown.

Considering the top or compression flange first, which is shown to the left of CD. The moment of resistance of the section of the two main angles is—

$$\begin{array}{ccccc} \text{Area.} & \text{Effective depth.} & \text{Stress.} & & \\ 5 & \times & 42 & \times & 4 \\ & & & & = 840 \text{ inch-tons} = 70 \text{ ft.-tons.} \end{array}$$

The moment of resistance of the section of each plate is—

$$\begin{aligned} 10 \times 42 \times 4 &= 1680 \text{ inch-tons.} \\ &= 140 \text{ ft.-tons.} \end{aligned}$$

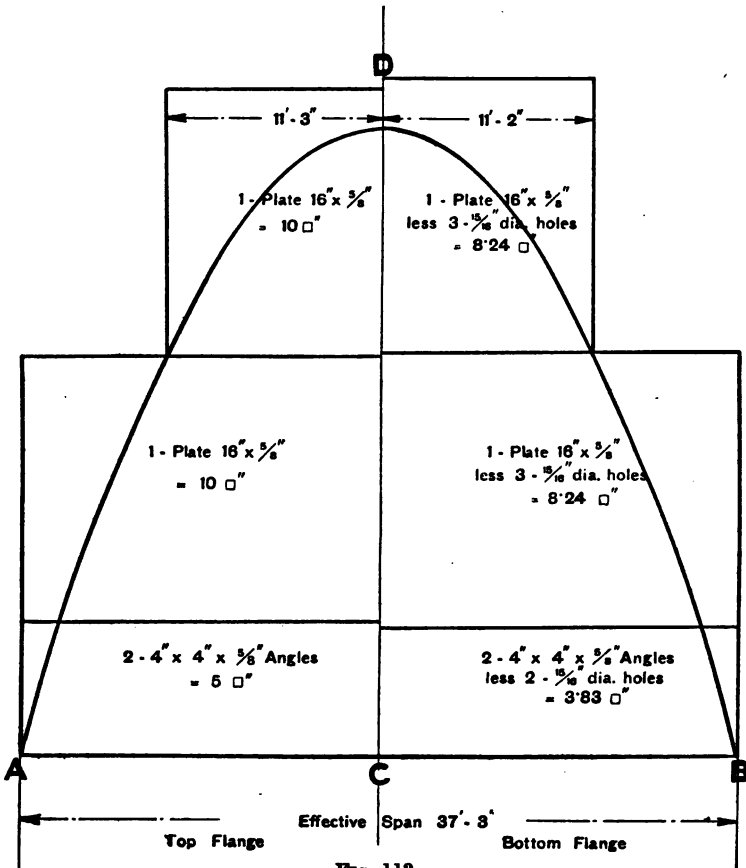


FIG. 113.

Similarly for the bottom or tension flange, which is shown to the right of CD.

The moment of resistance of the nett section of the two main angles, is—

$$\begin{aligned} 3.83 \times 42 \times 5 &= 804.30 \text{ inch-tons.} \\ &= 67.02 \text{ ft.-tons.} \end{aligned}$$

The moment of resistance of the nett section of each plate is $8.245 \times 42 \times 5 = 1731.45$ inch-tons = 144.28 ft.-tons. The points where the horizontal

lines cut the parabola give the theoretical lengths of the flange plates. For the reason explained on page 53, flange plates that do not extend the whole length of the girder, should have their theoretical length increased at each end.

The increase in length at each end is usually made such, that the resistance of the rivets in the additional length is equal to the resistance of the plate.

In the case under consideration the theoretical length of the second plate in the top flange scales 22' 6". The resistance of the plate in compression is $10 \times 4 = 40$ tons. A $\frac{3}{8}$ " dia. rivet in single shear at 4 tons per square inch = 2.3 tons, therefore the number of rivets required in the additional length at each end = say 17, which makes the total length of the plate 25' 8". The theoretical length of the second plate in the bottom flange scales 22' 4", and the resistance of the plate in tension is $8.245 \times 5 = 41.25$ tons, therefore the number of rivets required in the additional length at each end = say 17, as before, making the total length of the plate, 25' 8". The top and bottom flanges will therefore be the same.

Joints will not be required in the flange plates and main angles, as no difficulty will be experienced in obtaining plates and angles of the length required.

Design of Web Section.—In designing the web a vertical section of the girder over the abutment is taken for the purposes of calculation, because the maximum shearing force occurs at this section.

The shearing force at the ends due to the dead load is equal to the dead load reaction, which in this case is 10 tons.

The shearing force at the ends due to the live load is equal to the live load reaction, and from the diagram on page 56, the equivalent live load per foot run corresponding to the greatest shear for a span of 37' 3" is 3 tons, for a single line of railway. Therefore the live load on one main girder is $37.25 \times \frac{3}{2} =$ say 56 tons, and the shearing force at the ends due to the live load is $\frac{56}{2} = 28$ tons.

Maximum and Shear.—

Live Load = 28 tons.

Dead Load = 10 ,,

Total 38 ,,

The maximum stress to be allowed in the web is 3 tons per square inch.

Nett area of vertical web section required at the ends is $\frac{38}{3} = 12.70$ square inches.

The effective depth of the web (i.e. the depth minus the total lengths of the diameters of the rivet holes occurring at the section) for preliminary

calculations may be taken as equal to $\frac{3}{4}$ of the total depth, therefore the effective depth of the web in this case $= 42 \times \frac{3}{4} = 31.5$ inches.

From the above, the web thickness required

$$= \frac{\text{Total maximum shearing force in tons}}{\text{effective depth of web in inches} \times \text{allowable shearing stress in tons per square inch}}$$

$$= \frac{38}{31.5 \times 3} = .40, \text{ say } \frac{7}{16}'' \text{ thick.}$$

As stated in Chapter III. the section of the web, as calculated to resist this shearing force alone, is nearly always too small, the criterion of web design in most cases being the bearing resistance of the rivets connecting the web plate to the main angles, so in the following calculations for these rivets it will be seen whether the thickness calculated above is sufficient.

Riveting of Main Angles to Web.—

Total shear at ends = 38 tons.

Shear per foot run at ends $= \frac{38}{3.5} = 10.9$ tons.

Shearing value of rivets 4 tons per square inch.

Bearing „ „ 8 „ „

The rivets are in double shear, and for $\frac{7}{8}''$ dia. rivets this value is $0.601 \times 4 \times 2 = 4.8$ tons per rivet, and the bearing stress value with a web $\frac{7}{8}''$ thick $= \frac{7}{8} \times \frac{7}{8} \times 8 = 3.06$ tons per rivet. The bearing value of the rivets is the lowest, therefore the number of rivets required per foot run at the ends $= \frac{10.9}{3.06} = 3.56$. This would mean the rivets being 3" pitch at

the ends, and as the shearing force decreases towards the centre of the girder the pitch of the rivets could be increased at a certain distance from each end of the girder, the point of change depending upon the shearing force.

By increasing the web plate to $\frac{1}{2}''$ thick, the bearing value of each rivet is increased to 3.5 tons, the number of rivets required per foot run now being $\frac{10.9}{3.5} =$ say 3, or 4" pitch, so there are two alternatives, the web plate can either be $\frac{7}{8}''$ thick with rivets 3" pitch at the ends increased to 4" pitch towards the centre of the girder, or $\frac{1}{2}''$ thick with rivets 4" pitch throughout.

As it is desirable to keep the pitch and diameter of the rivets constant throughout, if possible, it will be seen that in this case the most economical method is to slightly increase the calculated thickness of the web plate rather than vary the pitch of the rivets.

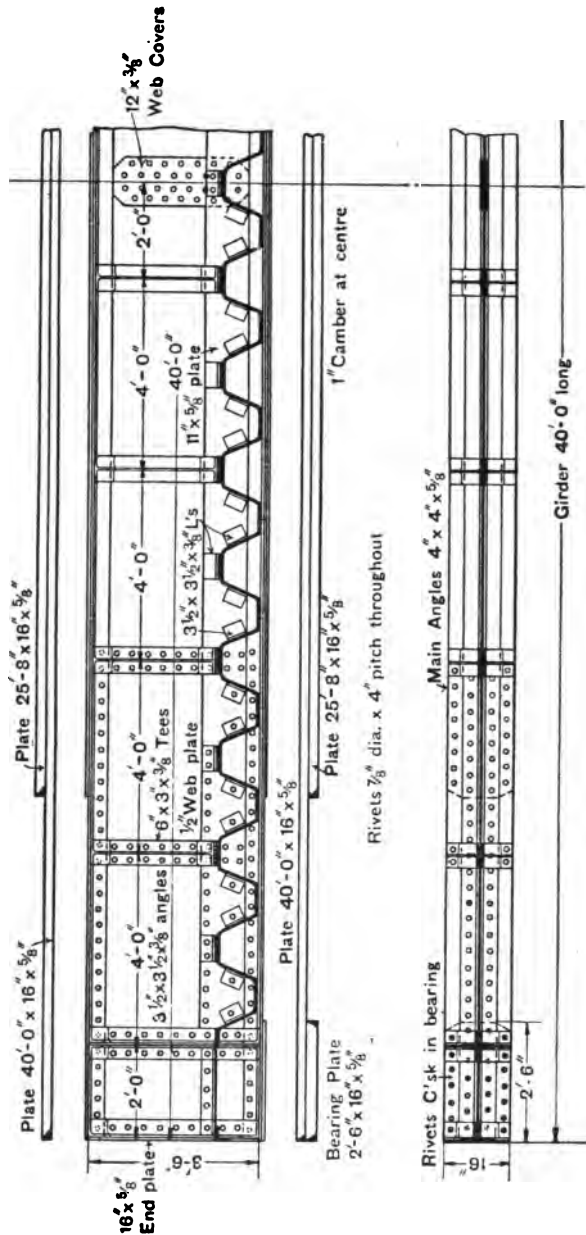


FIG. 114.

The web plate will therefore be $\frac{1}{2}''$ thick, and the rivets connecting the main angles to the web and flanges $\frac{7}{8}''$ diameter $\times$ 4" pitch throughout. No

calculations are necessary for the rivets connecting the main angles to the flanges, as for practical reasons these are spaced to suit those through the web, the number provided being greatly in excess of that required.

Design of Web Joint.—A web plate 40' 0" long $\times$ 3' 6" deep, would be too long to handle conveniently in the yard, so it will be better to provide a joint, and this will be best at the centre of the girder where the shearing force is a minimum.

It has been shown in Chapter VI., that in a web plate joint, the shearing resistance of the rivets must be equal to the net sectional area of the web. When the area of the metal in the various rivet sections is just equal to the net area of the web section, it follows that the joint is just equal in strength to an unbroken web. The web would shear and the rivets between the covers and web would shear at the same time.

$$\begin{aligned}\text{Net area of web plate} &= \frac{3}{4}(42" \times \frac{1}{2}) \\ &= 15.75 \text{ square inches.}\end{aligned}$$

Each $\frac{7}{8}"$ rivet in double shear has an area of $0.61 \times 2 = 1.22$ square inches, and the area of the rivets connecting the web covers to the web must be just equal to the net area of metal in the web at the joint.

$$\therefore \text{Number of rivets required} = \frac{15.75}{1.22} = 12.9 \text{ or } 13$$

The greatest number of rivets that can be put in a vertical section of the web, between the top and bottom main angles at 4" pitch is 7, therefore a double riveted web cover will be necessary. A cover must be provided on each side of the web, and following the usual rule for cover plates, each web cover plate is made $\frac{3}{4}$ of the thickness of the web plate, so in this case the cover plates will be $\frac{3}{8}"$ thick $\times$ 12" wide, as shown in Fig. 114.

Girder Bearings.—Each main girder will be provided with steel bearing plates $\frac{5}{8}"$ thick, and these will rest on cast-iron bedplates, as shown in Fig. 115.

The allowable pressure on the girder bedstones must not exceed 12 tons per square foot.

The greatest load at the end of each girder will be as follows :—

Live load	...	...	28 tons.
Dead load	...	...	10 "
Total	...	...	38 "

$$\text{Area of bedplate} = \frac{38}{12} = 3.17 \text{ square feet.}$$

The width of the flange plate is 16" and the bedplate will be 3" wider at each side, making the width of the latter 1' 10". At the commencement of

these calculations the length of the bearing plate was assumed to be 2' 9" long, but this length does not suit the pitch of the rivets, so it will be made 2' 6" long. This gives an area of $2' 6" \times 1' 10" = 4.57$ square feet, and a pressure on the bedstone of $\frac{38}{4.57} = 8.32$ tons per square foot, which is well within the pressure to be allowed. At the same time the length of the bedplate does not exceed $1\frac{1}{2}$ times its width. The bearing plates will be $\frac{5}{8}"$ thick and riveted to the underside of the bottom flange plate with counter-sunk rivets. The main girders will be secured to the bedplate by means of tap bolts, the holes through the girder being slotted at one end to allow of contraction and expansion, those in the bedplate being drilled and tapped.

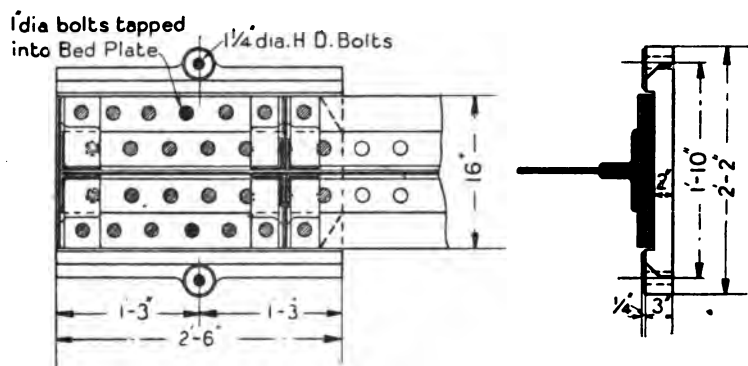


FIG. 115.

The cast-iron bedplate will be fastened to the masonry by holding-down bolts so as to prevent any lateral movement of the superstructure.

Stiffeners.—In this case the position of the stiffeners is fixed by practical considerations, *i.e.* the pitch of the troughs, and this being 2 ft., it will be best to have a stiffener every 4 ft., commencing at 2 ft. each side the centre of the girder. The stiffeners will be composed of $6" \times 3" \times \frac{3}{8}"$ tees, except those over the bearing plate which will be gusset stiffeners, and composed of $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{3}{8}"$ angles with gusset plates $\frac{3}{8}"$ thick.

The stiffeners over the troughs will be stopped $\frac{1}{2}"$ short of the top of the troughs, so that during erection the troughs can be easily slid into position, after which a $\frac{1}{2}"$ packing is inserted, and the stiffeners riveted to the troughs.

Waterproofing and Drainage.—The troughs are filled with bituminous concrete (a mixture of broken stone and boiling pitch) to the level of their upper surfaces, and to some extent above them at the centre of the bridge, forming a summit, and the surface graded so as to give a good fall to each end of the bridge, the camber of the main girders also assisting in this respect.

The concrete is then covered over with a layer of natural rock asphalt $1\frac{1}{2}$ " thick, this forming an impervious surface. The ends of the troughs are finished as shown in Fig. 116. A drip angle, riveted to the underside of the

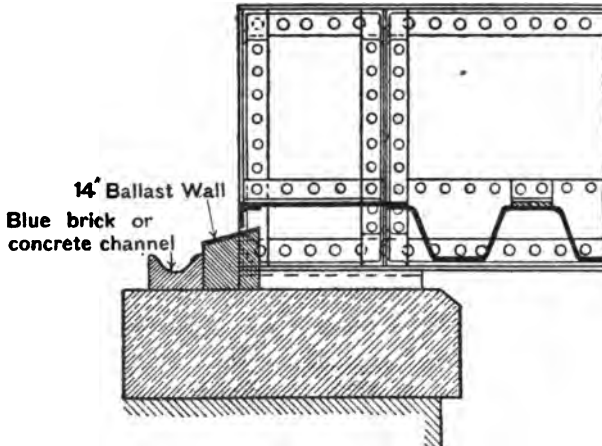


FIG. 116.

floor plate, projects over a dwarf wall which carries the water into concrete or blue-brick channels, discharging to the sides of the embankment or to down-pipes. The dwarf wall also keeps the ballast from the bedstones and the steelwork.

EXAMPLE II

DESIGN FOR AN UNDERBRIDGE OF THE THREE-GIRDER "THROUGH" TYPE WITH CROSS-GIRDERS, RAIL-BEARERS AND PLATED FLOOR, TO CARRY A DOUBLE LINE OF RAILWAY OVER A SQUARE SPAN OF 40 FEET

Assumed Data.—Live load. Type locomotive. See diagram, page 56.

Allowable Stresses.—When the live load is more than half the total load the allowable stresses per square inch to be as follows :—

Stress in Flanges.

$$3\frac{3}{4} \times \frac{\text{Live load} + \text{Dead load}}{\text{Live load}}$$

Stress in Rivets.

$$\text{Shear} \quad 2\frac{1}{4} \times \frac{\text{Live load} + \text{Dead load}}{\text{Live load}}$$

$$\text{Bearing} \quad 6 \times \frac{\text{Live load} + \text{Dead load}}{\text{Live load}}$$

Stress in webs not to exceed 3·5 tons per square inch.

Net areas to be taken for both tension and compression members of girders. The area of the flanges for calculation is to be the net area of the angle bars plus the net area of the flange plates.

All rivet holes to be taken as $\frac{1}{8}$ " larger diameter than that of the rivet.

All girders to camber 1 inch in 40 feet.

The allowable pressure on girder beds not to exceed 12 tons per square foot, and when the live load is more than half the total load, the allowable load on each square foot

$$= 6 \times \frac{\text{Live load} + \text{Dead load}}{\text{Live load}}$$

Floor of bridge to be watertight, and a close parapet to be provided on each side of the bridge.

Rail Bearers.—*Effective Span, 8' 0" ; Effective Depth, 1' 0".*

Commencing with the design of the floor system, and assuming the main girders to be spaced 11' 6" centres, and the flanges to be 18" wide, the width of the floor plating between the main girders will be 10' 0". The approximate dead loads on one rail-bearer are as follows :—

$\frac{3}{8}$ " Floor plates 40 square ft. @ 15·3 lbs.	= 612 lbs.
2" Cement concrete, say 6·64 c. ft. @ 120 lbs.	= 800 „
1" Asphalt, say 3·5 c. ft. @ 156 lbs.	= 546 „
7" Ballast, say 23·5 c. ft. @ 120 lbs.	= 2820 „
Permanent way $8 \times \frac{175}{2}$	= 700 „
Rail-bearers own weight	say 1000 „
	6478 = say 3 tons.

The live load on one rail-bearer obtained from diagram on page 56,
 $= \frac{40}{2} = 20$ tons.

Total loads = 20 + 3 = 23 tons uniformly distributed.

As the live load is more than half the total load the allowable flange stress

$$= 3\frac{1}{4} \times \frac{20 + 3}{20} = 4·31 \text{ tons per square inch.}$$

Total bending moment at centre of rail-bearer = $\frac{23 \times 8}{8} = 23$ foot-tons.

The effective depth is 1 ft., therefore the net area required in each flange

$$= \frac{\text{Bending moment}}{\text{Allowable stress} \times \text{Effective depth}} = \frac{23}{4·31 \times 1} = 5·33 \text{ square inches.}$$

The composition of the flanges will be as follows :—

Bottom or Tension Flange.—Assuming that $\frac{7}{8}$ " dia. rivets are required in the main angles, these latter must be at least $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{1}{2}"$, and a suitable width of flange plate for these would be 8".

$$\begin{aligned} 1 \text{ Plate } 8" \times \frac{1}{2}" \text{ less } 2 \frac{1\frac{5}{8}}{8}" \text{ dia. holes} &= 3.06 \text{ sq. ins.} \\ 2 \text{ Angles } 3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{1}{2}" \text{ less } 4 \frac{1\frac{5}{8}}{8}" \text{ dia. holes} &= 4.62 \text{ ,,} \\ &7.68 \text{ ,,} \end{aligned}$$

Top or Compression Flange.—As the floor plates are connected to the top flange plate, this must be made wider than the bottom flange to the extent of 5", making the plate 13" wide.

$$\begin{aligned} 1 \text{ Plate } 13" \times \frac{1}{2}" \text{ less } 2 \frac{1\frac{3}{8}}{8}" \text{ and } 2 \frac{1\frac{5}{8}}{8}" \text{ dia. holes} &= 4.75 \text{ sq. ins.} \\ 2 \text{ Angles } 3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{1}{2}" \text{ less } 4 \frac{1\frac{5}{8}}{8}" \text{ dia. holes} &= 4.62 \text{ ,,} \\ &9.37 \text{ ,,} \end{aligned}$$

Shear at Ends.

$$\left. \begin{array}{l} \text{Shear due to live load} \\ \text{(See diagram, page 56.)} \end{array} \right\} = \frac{21.25}{2} = 10.62 \text{ tons.}$$

$$\begin{aligned} \text{Shear due to dead load} &= \frac{3}{2} = 1.50 \text{ ,,} \\ &12.12 \text{ ,,} \end{aligned}$$

Riveting of Main Angles to Web.

$$\begin{aligned} \text{Shear at ends} &= 12.12 \text{ tons.} \\ \text{Shear per foot run} &= 12.12 \text{ tons.} \end{aligned}$$

The allowable stresses on the rivets are as follows :—

$$\text{Shear } 2.75 \times \frac{24.24}{21.25} = 3.13 \text{ tons per square inch.}$$

$$\text{Bearing } 6 \times \frac{24.24}{21.25} = 6.84 \text{ ,, ,}$$

These rivets are in double shear, and for $\frac{7}{8}$ " diameter rivets this value is $.601 \times 3.13 \times 2 = 3.76$ tons per rivet, and the bearing stress value assuming the web plate to be $\frac{1}{2}"$ thick is $\frac{7}{8}" \times \frac{1}{2}" \times 6.84 = 2.99$ tons per rivet.

The latter value being the lowest, the number of rivets per foot run required at the ends is $\frac{12.12}{2.99} = \text{say } 4$, therefore the rivets will be 3" pitch at the ends. At 15" from each end of the rail-bearer the shear per foot run is 8.5 tons, and the number of rivets required per foot run $\frac{8.5}{2.99} = \text{say } 3$.

The rivets connecting the main angles to the web will be $\frac{7}{8}"$ diameter,

3" pitch at each end for a distance of 15" or four pitches, and 4" pitch in the intermediate length. The rivets connecting the main angles to the flange plates will be of the same diameter and pitched to suit those through the web.

Thickness of Web.—The effective depth of the web at the ends is $12'' - 3 \times \frac{1}{8}'' = 9'' \cdot 3$, so with a web plate $\frac{1}{2}$ in. thick the stress is $\frac{12 \cdot 12}{9 \cdot 3 \times 0 \cdot 5} = 2 \cdot 60$ tons per sq. in., which is within the limits permissible.

Connection of Rail-bearer to Cross-girder.—The rail-bearers will be connected to the cross-girders by means of angle cleats, and ample rivets should be provided in these connections, more than theoretically necessary, as these rivets are put in at site by hand.

The rivets through the cleats and cross-girder web are in single shear, while those through the rail-bearer are in double shear.

Shear at ends = 12·12 tons.

Number of rivets required through cross-girder = $\frac{12 \cdot 12}{1 \cdot 88} = 6 \cdot 4$, say 7

Number of rivets required through rail-bearer = $\frac{12 \cdot 12}{2 \cdot 99} =$ say 4

If the cleats are $6'' \times 6'' \times \frac{1}{2}''$ ample rivets will be provided.

Cross Girders.

<i>Effective Span</i> ...	... 11' 6"
<i>Effective Depth</i> ...	... 1' 1"

The dead load from the rail-bearers is 3 tons, i.e. half from one rail-bearer and half from another, acting at the points of attachment to the cross-girder, and the bending moment from these weights is $3 \times 3 \cdot 25 = 9 \cdot 75$ foot-tons. Assuming the weight of a cross-girder to be 1 ton, the bending moment from this cause is $\frac{1 \times 11 \cdot 5}{8} = 1 \cdot 44$ foot-tons. The greatest live load that can be brought on a cross-girder will be when the wheel loads are in the position shown in Fig. 117, and will be as follows:—

$$10 + 2\left(\frac{10 \times 0 \cdot 5}{8}\right) = 11 \cdot 25 \text{ tons.}$$

The bending moment due to the live load

$$= 11 \cdot 25 \times 3 \cdot 25 = 36 \cdot 56 \text{ foot-tons.}$$

The total bending moment due to the dead and live loads

$$= 9 \cdot 75 + 1 \cdot 44 + 36 \cdot 56 = 47 \cdot 75 \text{ foot-tons.}$$

The live load is more than half the total load, therefore the allowable flange stress

$$= 3\frac{1}{2} \times \frac{29.5}{22.5} = 4.90 \text{ tons per square inch.}$$

The area required in each flange

$$= \frac{47.75}{4.90 \times 1.08} = 9.03 \text{ square inches.}$$

The flanges will be composed as follows :—

Bottom or Tension Flange.

1 Plate $12'' \times \frac{1}{2}''$ less 2 $\frac{1}{8}''$ dia. holes	= 5.06 sq. inches.
2 Angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ less 4 $\frac{1}{8}''$ dia. holes	= 4.62 „
	9.68 „

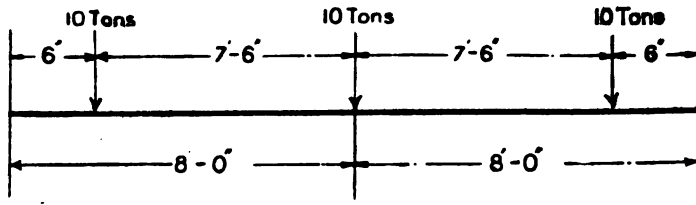


FIG. 117.

Top or Compression Flange.—The top flange plate must be wide enough to allow for the attachment of the floor plates.

1 Plate $13'' \times \frac{1}{2}''$ less 2 $\frac{1}{8}''$ and 2 $\frac{3}{4}''$ dia. holes	= 4.75 sq. inches.
2 Angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ less 4 $\frac{1}{8}''$ dia. holes	= 4.62 „
	9.37 „

Riveting of Main Angles to Web.

Shear at Ends.

$$\text{Live load} = 11.25 \text{ tons.}$$

$$\text{Dead loads} = 3.50 \text{ „}$$

$$\hline 14.75 \text{ „}$$

$$\text{Shear per foot run} = \frac{14.75}{1.08} = 13.65 \text{ tons.}$$

The allowable rivet stresses are as follows :—

$$\text{Shear } 2.75 \times \frac{14.75}{11.25} = 3.60 \text{ tons.}$$

$$\text{Bearing } 6 \times \frac{14.75}{11.25} = 7.86 \text{ „}$$

The shearing resistance of one $\frac{7}{8}$ " diameter rivet in double shear is $\cdot 601 \times 3 \cdot 60 \times 2 = 4 \cdot 32$ tons, and the bearing resistance, assuming the web plate to be $\frac{1}{2}$ " thick, is $\frac{7}{8}" \times \frac{1}{2}" \times 7 \cdot 86 = 3 \cdot 44$ tons.

The latter value being the lowest, the number of rivets per foot run required at the ends is $\frac{13 \cdot 65}{3 \cdot 44} =$ say 4, therefore the pitch will be 3", and this pitch will be carried through at each end to the rail-bearer connections. In the centre bay between these points, the only shear is that due to the dead load, which is very small, so in this portion the pitch will be increased to 4 inches.

The effective depth of the web at the ends is $13" - 4 \times \frac{1}{8}" = 9 \cdot 25$ inches with a web plate $\frac{1}{2}$ inch thick, the stress is $\frac{14 \cdot 75}{9 \cdot 25 \times 0 \cdot 5} = 3 \cdot 18$ tons per square inch.

Connection of Cross-girder to Main Girder.—The rivets are in single shear, and the number required $= \frac{14 \cdot 75}{2 \cdot 16} =$ say 7 rivets.

Using $6" \times 6" \times \frac{1}{2}"$ angle cleats ample rivets will be provided.

Centre Main Girder.—The clear span of the bridge is 40 ft., and for purposes of calculation the bearings may be taken at, say 2' 6" long, and these will be 6" from the face of the abutment at each end, making the total length of the girder 46' 0", and giving an effective span of 43' 6".

If the girder is made 4' 0" deep, this gives a proportion of almost one-eleventh of the span.

The uniformly distributed live load per foot run for a single line of railway for a span of 43' 6", as given by the diagram on page 56, is 2'46 tons, or a total distributed load on the girder of $43 \cdot 5 \times 2 \cdot 46 = 107$ tons.

The approximate dead loads on the girder are as follows:—

Floor plates, 435 square feet @ 15'3 lbs.	= 3'00 tons.
1" Asphalt, 36 cubic feet @ 156 lbs.	= 2'50 "
2" Cement, 72 " " @ 120 lbs.	= 3'85 "
Ballast, 252'3 " " @ 120 lbs.	= 13'51 "
Permanent way, $43 \cdot 5 \times 175$ lbs.	= 3'40 "
Rail-bearers, No. 11 @ 1000 lbs. each	= 5'00 "
Cross-girders, No. 5 (10 $\frac{1}{2}$ ones) @ 1 ton each	= 5'00 "
Main girders own weight, say	= 10'50 "
	46'76 = say 47 tons.

The live load being more than half the total load, the allowable stress per square inch $= 3 \frac{1}{2} \times \frac{107 + 47}{107} = 5 \cdot 4$ tons per square inch.

The total bending moment at the centre of the girder is

$$\frac{154 \times 43.5}{8} = \text{say } 837 \text{ ft.-tons.}$$

The effective depth is 4' 0", therefore the nett area required in each flange

$$= \frac{837}{5.4 \times 4} = 38.75 \text{ square inches.}$$

The width of the flange plates must be such that the structure gauge is not fouled, and from the diagram given in the Appendix, it will be seen that the distance from the centre of the track to the inner edge of the flange, must not be less than 4' 9".

The main girders are 11' 6" centre to centre, so if the flanges are made 1' 6" wide, the distance from the centre of the track to the inner edge of flange is 5' 0", which gives ample clearance.

The composition of the flanges will be the same both for tension and compression as follows :—

2 Angles, 6" × 4" × $\frac{5}{8}$ " less 6 $\frac{1}{2}$ " dia. holes	=	8.22 sq. inches.
2 Plates, 18" × $\frac{5}{8}$ " each less 3 $\frac{1}{8}$ "	„	= 18.99 „
2 Plates, 18" × $\frac{1}{2}$ " each less 3 $\frac{1}{8}$ "	„	= 15.19 „
		42.40 „

The first flange plate in both the top and bottom flanges, will, of course, extend for the whole length of the girder, but the lengths of the remaining flange plate will be curtailed, as explained on page 28, and as shown in Fig. 118.

Riveting of Main Angles to Web.

$$\left. \begin{array}{l} \text{Shear at ends due to live load} \\ \text{(See diagram, page 56)} \end{array} \right\} = \frac{126.15}{2} = 63.07 \text{ tons.}$$

$$\text{Shear at ends due to dead load} = \frac{47}{2} = 23.50 \text{ „}$$

$$\text{Total shear} = \underline{86.57 \text{ „}}$$

$$\text{Shear per foot run at ends} = \frac{86.57}{4} = 21.64 \text{ tons.}$$

The allowable stresses on the rivets are as follows :—

$$\text{Shear } 2.75 \times \frac{86.57}{63.07} = 3.76 \text{ tons per square inch.}$$

$$\text{Bearing } 6 \times \frac{86.57}{63.07} = 8.22 \text{ „ „}$$

These rivets are in double shear, and for $\frac{7}{8}$ " dia. rivets this value is

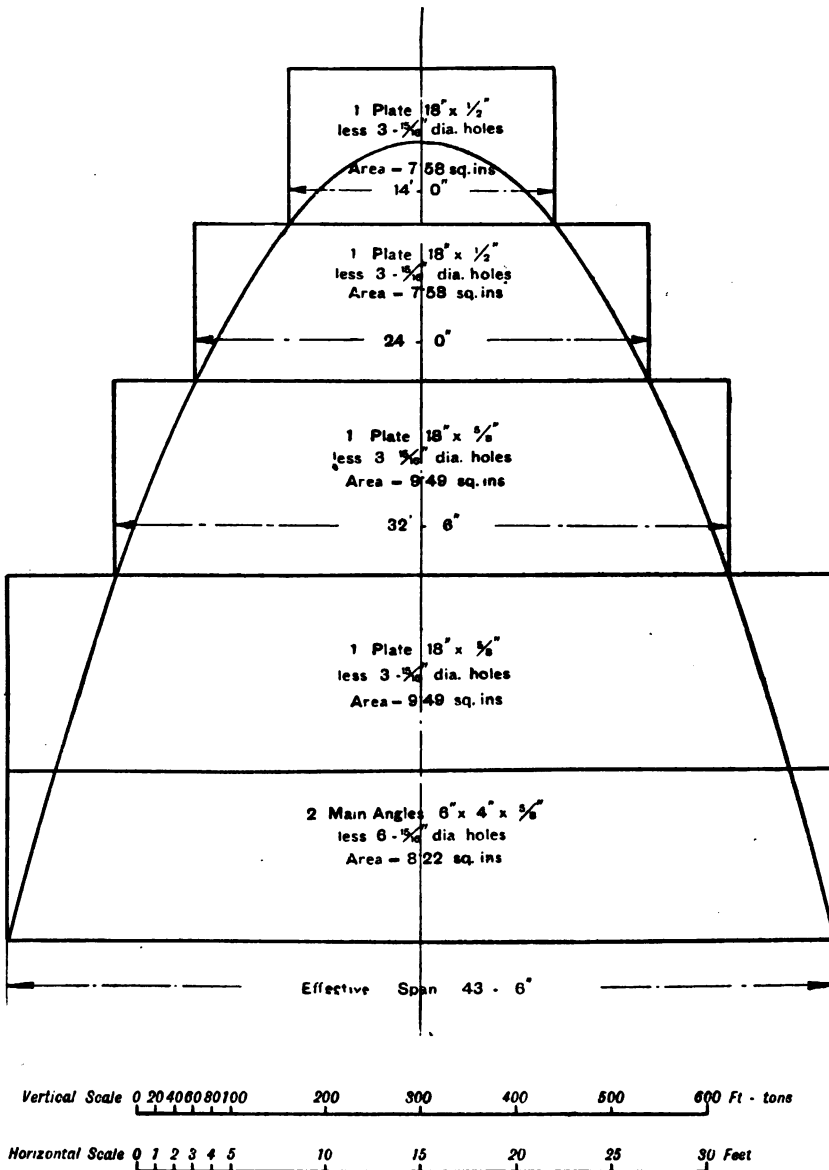


FIG. 118.

$\cdot 601 \times 3.76 \times 2 = 4.52$ tons per rivet, and the bearing stress value assuming the web at the ends to be $\frac{5}{8}$ " thick is $\frac{7}{8}" \times \frac{5}{8}" \times 8.22 = 4.5$ tons per rivet. In

this case the bearing value of the rivets is the lowest, therefore the number of rivets per foot run required at the ends is $\frac{21.64}{4.5} = \text{say } 5 \text{ rivets.}$

The rivets connecting the main angles to the web will be $\frac{7}{8}$ " diameter, double riveted at 4" pitch at the ends. At 9' 0" from each end of the girder the shear per foot run is 14.5 tons, and the number of rivets required per foot run 3.22, so in the central portion of the girder the pitch of the rivets can be increased. The inner rows of rivets can be increased to 8" pitch, but in order to ensure tight work between the main angles and plates, the outer rows of rivets will be kept 4" pitch throughout.

Design of Web Section.

Total shear at ends = 86.57 tons.

Effective depth of webs at ends = $48" - 11 \times \frac{5}{8}" = 37.69"$.

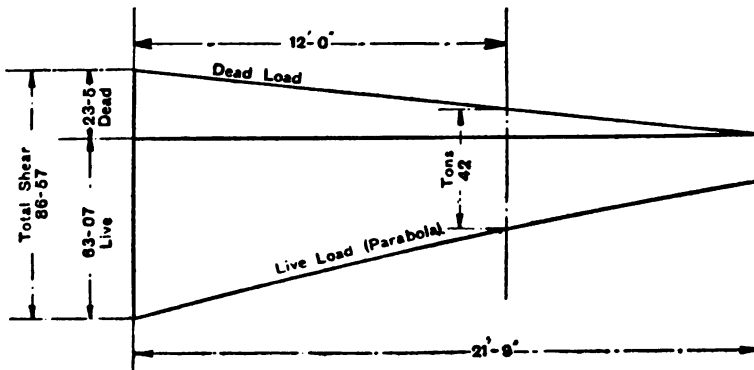


FIG. 119.

With a web plate $\frac{5}{8}$ " thick the stress = $\frac{86.57}{37.69 \times .625} = 3.68 \text{ tons per square inch.}$

At 13' 0" from the ends of the girder the shear is 42 tons, scaled from the shearing force diagram, Fig. 119, and at these points it will be best to joint the web plates, at the same time reducing the thickness of the central portion.

If this central portion of the web is made $\frac{1}{2}$ " thick, the web stress = $\frac{42}{37.69 \times .5} = \text{say } 2.23 \text{ tons per square inch.}$

Design of Web Joint.—The shearing resistance of the rivets in the joint must be equal to the net sectional area of the webs.

Net area of $\frac{5}{8}$ " web plate = 23.56 square inches.

Each $\frac{7}{8}$ " rivet in double shear has an area of 1.20 square inches.

Number of rivets required $\frac{23.56}{1.20} = \text{say } 19 \text{ rivets.}$

With a double-riveted web cover, and a vertical pitch of 4", 18 rivets can be provided on each side of the joint.

A cover will be provided on each side of the joint, and each cover is made $\frac{3}{4}$ of the web thickness, say $\frac{5}{8}$ " thick.

Joints in Flange Plates.—The girders being 46' 0" long over all, it will be best to provide a joint in the first plate of the top and bottom flanges. These joints will not be opposite each other, but 4' 0" each side of the centre respectively.

The net area of an 18" $\times$ $\frac{5}{8}$ " plate = 9.49 square inches, and its resistance at 5.4 tons per square inch = 51.25 tons.

With a cover plate 3' 0" long, and cover strips on the inside of the flange plate, there will be 8 rivets in double shear and 8 rivets in single shear on each side of the joint.

The allowable working stress of rivets = $2\frac{3}{4} \times \frac{107+47}{107}$ = say 4 tons per square inch.

Resistance of $\frac{7}{8}$ " diameter rivet in single shear = $.601 \times 4$
= 2.4 tons.

The total resistance of the rivets provided on each side of the joint is as follows :—

Single shear	...	...	8×2.4	= 19.2 tons.
Double shear	...	...	$8 \times 2.4 \times 2$	= 38.4 ,,
				57.6 ,,

The combined thickness of the cover plates should be 50% greater than that of the plate covered, so these will be made $\frac{1}{2}$ " thick.

Joints in Main Angles.—The net area of a 6" $\times$ 4" $\times$ $\frac{5}{8}$ " angle = 4.11 square inches, and its resistance at 5.4 tons per square inch = 22.2 tons.

With a cover 2' 0" long, there will be 5 rivets in double shear and 3 rivets in single shear on each side of the joint.

The total resistance of these rivets is as follows :—

Single shear	...	...	3×2.4	= 7.2 tons.
Double shear	...	...	$5 \times 2.4 \times 2$	= 24.0 ,,
				31.2 ,,

The joints will be "staggered," i.e. the joints in the near and far angles of the top flange will correspond in position with, and break joint with, those in the far and near angles of the bottom flange, each joint being covered separately with double covers.

The covers will be $\frac{1}{2}$ " thick.

Girder Bearings.—Each girder will be provided with steel bearing plates $\frac{3}{4}$ " thick, resting on cast-iron bearings.

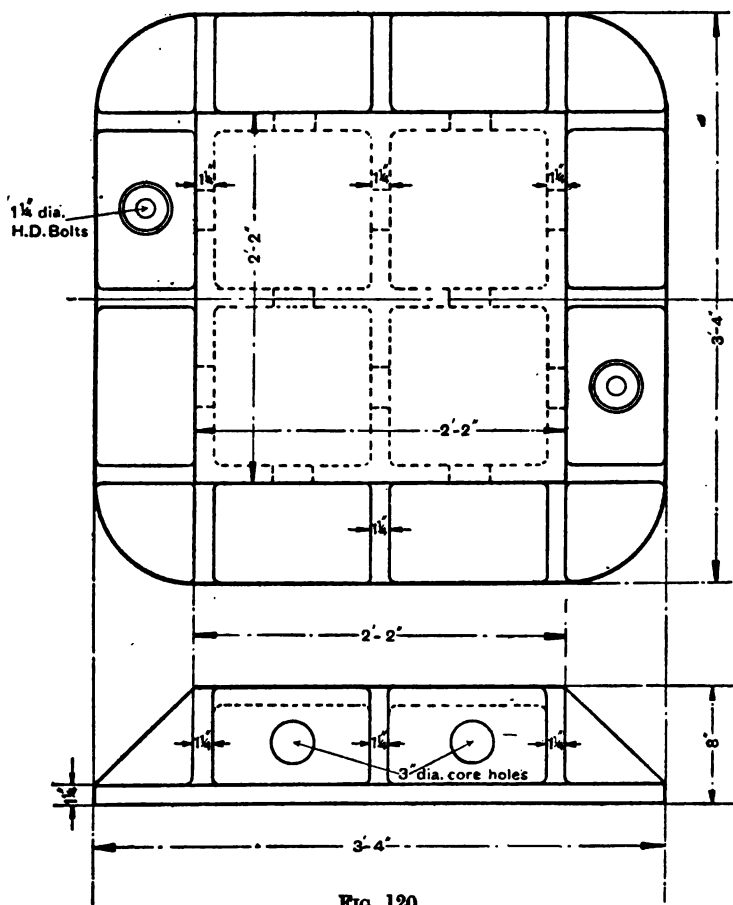


FIG. 120.

The reaction, or load, at each bearing of centre girder, is as follows :—

Live load	...	...	...	63.07 tons.
Dead load	...	...	...	23.50 "
				86.57 "

Allowable pressure on girder bedstones = $6 \times \frac{86.57}{63.07} = 8.22$ tons per square foot.

Area required = $\frac{86.57}{8.22} = 10.53$ square feet.

The steel bearing plate will be 2' 2" long, and of the same width, and the base of the cast-iron bearings 3' 4" square, giving a bearing area on the bedstones of 11 square feet. The girders will be fastened to the casting by 4 1" diameter tap bolts at each end, and to allow for the expansion and contraction due to changes of temperature, elongated holes will be provided in the girder at one end. The castings will be secured to the bedstones by means of Lewis bolts. Full details of these bearings are given in Fig. 120.

Outside Main Girders.—These girders will be of the same length and depth as the centre girder, therefore the effective span will be the same, viz. 43' 6".

The uniformly distributed live load for a single line of railway, for a span of 43' 6", as given by the diagram on page 56 is 2.46 tons per foot run, and each outer girder will carry one-half of this load, or 1.23 tons per foot run, giving a total distributed load on the girder of 53.5 tons.

The approximate dead loads on the girders are as follows :—

Floor plates	218 sq. ft.	@ 15.3 lbs. per sq. ft.	= 1.48 tons.
1" Asphalt	18 c. ft.	@ 156 lbs. per c. ft.	= 1.25 "
2" Cement	36 c. ft.	@ 120 lbs. " "	= 1.92 "
Ballast	126.6 c. ft.	@ 120 lbs. " "	= 6.75 "
Permanent way 43.5 ft. run	@ $1\frac{1}{2}$ lbs. per ft.		= 1.70 "
Rail-bearers	5½	@ 1000 lbs. each	= 2.50 "
Cross-girders	5	@ 1120 lbs. " "	= 2.50 "
Main girder and parapet	@	say	= 8.50 "
Say 27 tons.			26.66 "

The live load is more than half the total load, therefore the allowable stress per square inch = $3\frac{1}{2} \times \frac{53.5 + 27}{53.5} = 5.64$ tons per square inch.

The total bending moment at the centre of the girder is

$$\frac{80.5 \times 43.5}{8} = \text{say } 438 \text{ ft.-tons.}$$

The effective depth of the girder is 4' 0", therefore the net area required in each flange

$$= \frac{438}{5.64 \times 4} = 19.4 \text{ sq. inches.}$$

The composition of the flanges will be as follows :—

Bottom or Compression Flange.

2 Angles, 4" × 4" × ½" less 4 ½" dia. holes	= 5.625 sq. inches.
2 Plates, 18" × ½" each less 3 ½" " "	= 15.19 "
	20.815 "

Top or Tension Flange.

2 Angles, $4" \times 4" \times \frac{1}{2}"$	less 4 $\frac{15}{16}"$ dia. holes	=	5.625 sq. inches.
1 Plate, $21" \times \frac{1}{2}"$	,, 4 $\frac{15}{16}"$	=	8.625 ,,
1 Plate, $18" \times \frac{3}{8}"$	,, 3 $\frac{15}{16}"$	=	5.70 ,,
			<u>19.950</u> ,,

The first plate in the top flange is made 3" wider than the other plate so

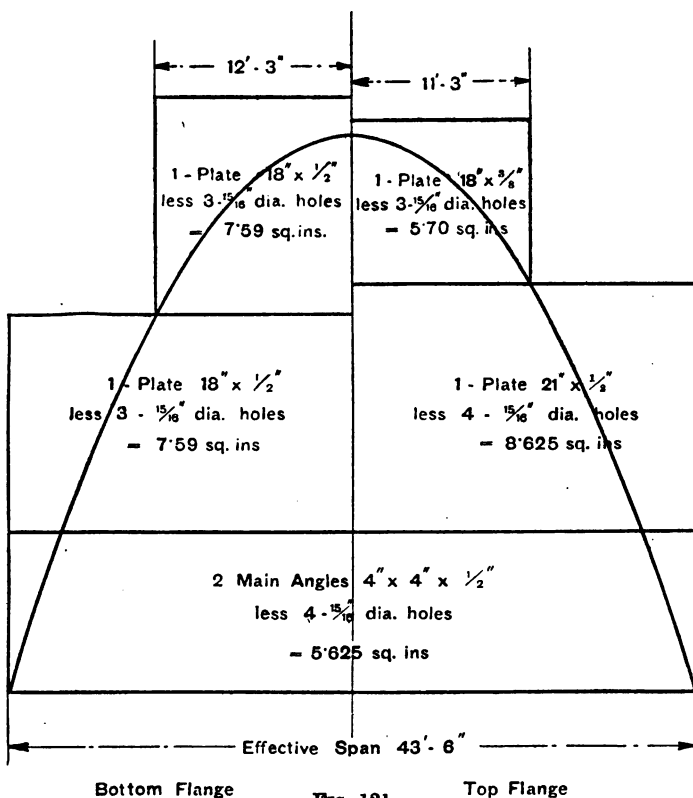


FIG. 121.

as to enable a connection to be made with the plates of the projecting platform.

The second flange plate in both the top and bottom flanges will be curtailed as shown in Fig. 121.

Riveting of Main Angles to Webs.—

Shear at ends due to live load	=	31.6 tons.
Shear at ends due to dead load	=	13.5 ,,
Total shear	=	<u>45.1</u> ,,

$$\text{Shear per foot run at ends} = \frac{45.1}{4} = 11.3 \text{ tons.}$$

The allowable stresses on the rivets are as follows:—

$$\text{Shear } 2.75 \times \frac{45.1}{31.6} = \text{say } 4 \text{ tons per sq. in.}$$

$$\text{Bearing } 6 \times \frac{45.1}{31.6} = \text{say } 8.5 \text{ tons per sq. in.}$$

The rivets are in double shear, and for $\frac{7}{8}$ " dia. rivets, this value is $0.601 \times 4 \times 2 = 4.8$ tons per rivet, and the bearing stress value, assuming the web at the ends to be $\frac{1}{2}$ " thick, is $\frac{7}{8} \times \frac{1}{2} \times 8.5 = 3.7$ tons. The latter value is the lowest, and is the criterion of strength, therefore the number of rivets per foot run required at the ends is $\frac{11.3}{3.7} = \text{say } 3$, so that $\frac{7}{8}$ " dia. rivets at 4" pitch throughout will be ample. The rivets connecting the main angles to the flanges will be spaced to suit.

Design of Web Section.—

Total shear at ends of girder = 45 tons.

Effective depth of webs at ends = 35.8 square inches.

With a web plate $\frac{1}{2}$ " thick, the stress

$$= \frac{45}{35.8 \times .5} = 2.5 \text{ tons per sq. in.}$$

It will be best to joint the web plates at 13' 0" from each end of the girder, at the same time reducing the thickness of the central portion to $\frac{3}{8}$ ", which will be of ample thickness.

Design of Web Joint.—As before, the shearing resistance of the rivets in the joint must be equal to the net sectional area of the web plate.

Net area of $\frac{1}{2}$ " web plate = 17.9 sq. ins.

Each $\frac{7}{8}$ " dia. rivet in double shear has a shearing area of 1.20 sq. ins., therefore the number of rivets required on each side of the joint

$$= \frac{17.9}{1.20} = \text{say } 15.$$

With a double-riveted web cover and a vertical pitch of 4", 19 rivets can be provided on each side of the joint.

Cover plates $\frac{1}{2}$ " thick will be provided on each side of the joint.

Joints in Flange Plates.—The first plate of the top and bottom flanges will be jointed at 4' 0" each side the centre of the girder respectively.

Joint in Top Flange Plate.—

Net area of $21'' \times \frac{1}{2}''$ plate = 8.625 square inches.

Resistance at 5.64 tons per square inch = 48.6 tons.

Allowable working stress for rivets = $2\frac{3}{4} \times \frac{53.5 + 27}{53.5} = 4.13$ tons per square inch.

With a cover plate $3' 0''$ long, and cover strips on the inside of the flange plate, there will be 8 rivets in single shear, and 8 rivets in double shear, and $5 \frac{3}{4}''$ diameter rivets in double shear, giving a total rivet resistance on each side of the joint of 68.5 tons, against 48.6 tons required.

The cover plate and strips will be $\frac{1}{2}''$ thick.

Joint in Bottom Flange Plate.—

Net area of $18'' \times \frac{1}{2}''$ plate = 7.59 square inches.

Resistance of plate = 42.8 tons.

With a cover plate $3' 0''$ long, and cover strips to suit, there will be a total rivet resistance on each side of the joint of 60 tons. The cover plates and strips will be $\frac{1}{2}''$ thick, and similar to those in the top flange.

Joints in Main Angles.—

Net area, of $4'' \times 4'' \times \frac{1}{2}''$ angle = 2.81 square inches.

Resistance at 5.64 tons per sq. in. = 15.84 tons.

With covers $2' 0''$ long, there will be 3 rivets in double shear and 2 rivets in single shear on each side of the joint, giving a total resistance of, say 20 tons.

The joints will be "staggered" similar to those in the centre girder, each joint being covered separately, with double covers, $\frac{1}{2}''$ thick.

Girder Bearings.—Each girder will be provided with steel bearing plates, $\frac{3}{4}''$ thick, resting on cast-iron bearings.

The reaction, or load, at each bearing is as follows :—

Live load = 31.6 tons.

Dead load = 13.5 "

Total 45.1 "

Allowable pressure on girder bedstones

$$= 6 \times \frac{45.1}{31.6} = 8.5 \text{ tons per sq. ft.}$$

$$\text{Area required} = \frac{45.1}{8.5} = 5.30 \text{ sq. ft.}$$

The steel bearing plate will be $2' 2''$ long and the same width as the flange

plate, viz. 1' 6", and the base of the cast-iron bearings 3' 0" long $\times$ 2' 6" wide, giving a bearing area on the bedstones of 7.5 sq. ft.

The girders and the cast-iron bearings will be fastened as in the centre girder.

Full details of these bearings are given in Fig. 122.

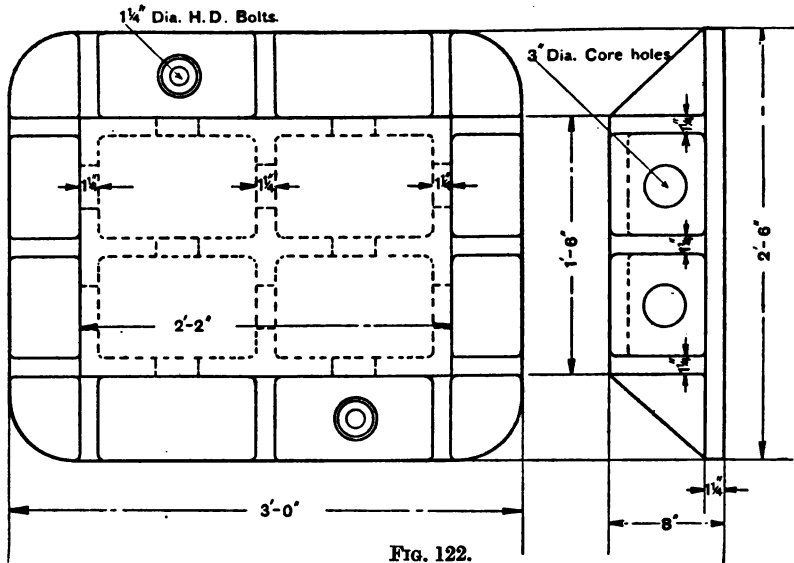


FIG. 122.

Waterproofing and Drainage.—The waterproofing and drainage of the bridge floor, and the protection of the top flanges of the main girders will be as described in Chapter VIII. and as shown on Plate I.

It should be noted that in the above calculations the dead loads have been more or less assumed, but the difference between these and the dead loads calculated from the complete design is not sufficient to warrant any alteration in the design.*

EXAMPLE III

DESIGN FOR 70-FT. CLEAR SPAN "DECK" SKEW BRIDGE DOUBLE LINE. ANGLE OF SKEW 45°

(4 Main Girders with Plate Flooring and Ballast)

Assumed Data.—

The bridge is assumed to be carrying a double line of railway over a river, the height from ordinary water-level to rail-level being 25 ft., and the clear headway under the bridge not less than 16 ft. See Fig. 123.

* The complete design for this bridge is shown on Plate I.

Live Load. Type Locomotive.—See diagram, page 56. Increment for Impact 80 per cent. See table, page 38.

Working Stresses.—

Tension and compression	$7\frac{1}{2}$ tons per sq. in.
Shear	$5\frac{1}{2}$ " "
Bearing	12 " "

Girders to camber 2 ins. at centre between bearings.

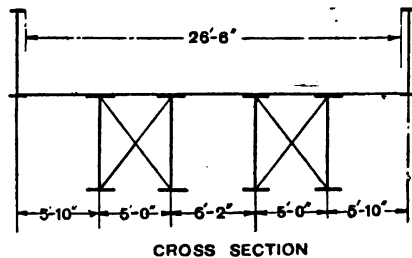
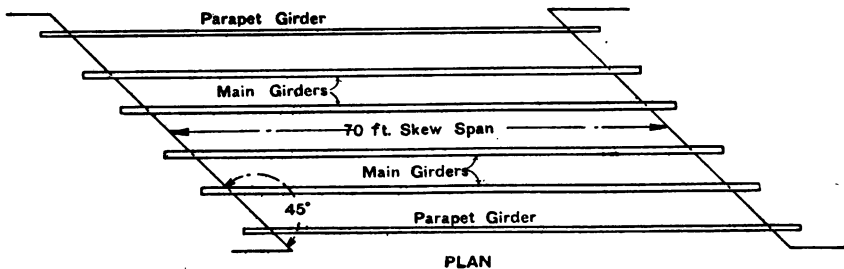


FIG. 123.

Dead Loads on Floor per lineal foot of Bridge.—

$\frac{3}{8}$ " Floor plates...	23' 0" $\times$ 1' 0" $\times$ 15.3 lbs. per sq. ft. =	352 lbs.
1" Asphalt	28' 0" $\times$ $\frac{1}{12}$ ' $\times$ 156 lbs. per c. ft. =	364 "
9" Ballast	26' 6" $\times$ $\frac{9}{12}$ ' $\times$ 120 lbs. per c. ft. =	2385 "
Permanent way (2 lines)	175 lbs. per ft. $\times$ 2	350 "
Say 3500 lbs. per lineal foot.		3451 "

Distribution of Dead Load on Girders.—This dead load per foot as above will be assumed to be distributed equally over the whole width

of bridge. Each girder, therefore, takes such proportion of this load as lies between the centre lines of the spaces to the left and right of the girder as shown in the diagram, Fig. 124. For instance, the girder "B" takes half

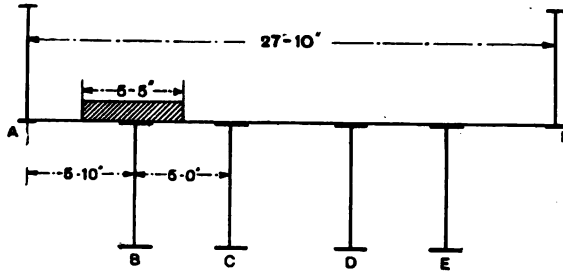


FIG. 124.

the portion of load lying between "A" and "B" plus half the portion between "B" and "C" = $\frac{5' 5''}{27' 10''} = 0.194$ of the total load (3500 lbs. per lin. ft. of span of the bridge).

The loads per lineal foot run of span calculated on this principle will be found as shown in Fig. 125.

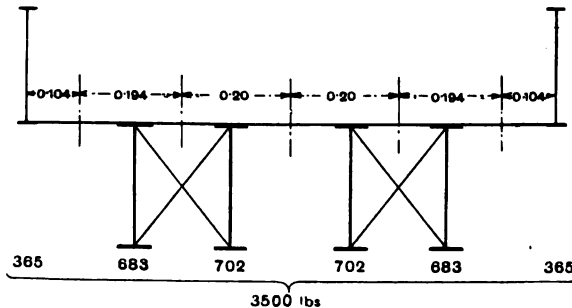


FIG. 125.

Weight of steelwork in each parapet girder will be, say 12 tons.

Weight of steelwork in each main girder will be, say 22 tons.

E.U.D. live load for 75' 6" span = 168 tons } See diagram, page 56.
for 2 lines = 336 ,, }

∴ Each main girder has a live load of $\frac{336}{4} = 84$ tons, and each parapet girder is designed to carry $\frac{1}{4}$ of this *live load* in order to provide for a derailment, in which case, an *additional load* would be imposed on it.

Design of Main Girders.—

Clear Span, 70 ft., Angle of Skew, 45°.—Before the overall length of the girders can be fixed in a skew span, it is necessary to make a sketch plan of the bearings, as shown in Fig. 126, as a greater "landing" is required for the girders with a skew span than for a square span, this is in order to provide satisfactory bearings.

This would be drawn to scale, and an approximate overall length can be obtained; it may be necessary to modify this later on, but the alteration would not be of sufficient importance to upset the calculations.

Span (centre to centre of bearings) 75' 6".

Overall length of girders 78' 0".

As there is ample construction depth for the bridge, the girders might

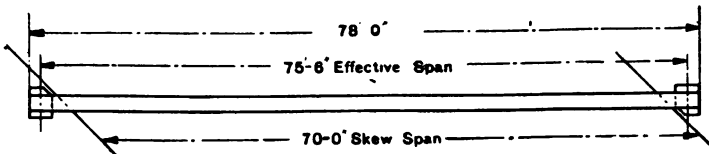


FIG. 126

be made their full economical depth = $\frac{1}{10}$ to $\frac{1}{12}$ of the span, say 6' 6" over angles.

	Due to B.M.	Due to S.F.
Live load per girder	= 84 tons.	93 tons.
Impact allowance (80 %)	= 67 "	74 "
Dead load (steelwork)	= 22 "	22 "
" { (Flooring, P.W. etc.,)	= 25 "	25 "
" { (702 × 75½)	= 198 "	214 "

In a long, deep girder of this type the load produced by wind pressure should be added to the B.M. live load. This load may be taken at 30 lbs. per sq. ft., of exposed surface of girder + 80 % allowance for impact.

Wind pressure = $75\frac{1}{2} \times 6\frac{1}{2} \times 30 = 14,730$ lbs.

Impact allowance (80 %) = 11,784 "

Total 26,514 "

Say 12 tons.

Maximum B.M. at centre of Main Girders.—

$$= \frac{198 \times 75\frac{1}{2}}{8}$$

$$= 1870 \text{ tons-feet.}$$

These lengths, with an additional amount added to provide for excess of B.M. at the haunches of the parabola, will be distributed equally on each side of the point A_1 as a centre line, and not B_1 , the geometrical centre line

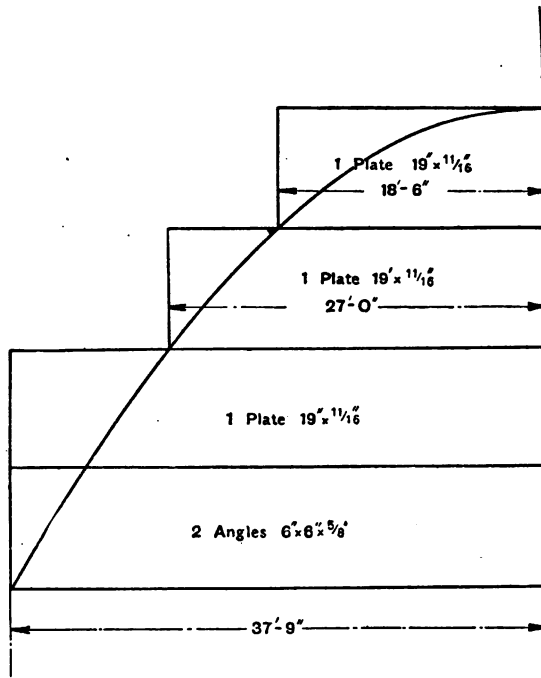


FIG. 128.

of the span. (See Fig. 129.) The reason for this is that the E.U.D. live loads used in the calculations are for *one line* of railway, and would, therefore, be considered to act at the centre line of the two rails. The maximum B.M.

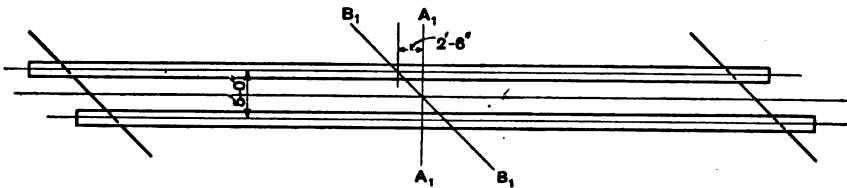


FIG. 129.

is, therefore, at the centre of the span on the centre line between the rails. This B.M. is transferred perpendicularly outwards to the girders by the flooring to the points A_1 and A_1 , which are taken as points of maximum B.M. for the girders.

Bearings.—

Steel bearing plate $2' 0'' \times 19'' \times \frac{3}{4}''$

$$\text{Maximum S.F. at ends} = \frac{214}{2} = 107 \text{ tons.}$$

C.I. Rocker Bearings.—

Area of C.I. bedplate = 8.7 sq. ft. (See Fig. 130.)

$$\text{Bearing pressure (including impact)} = \frac{107}{8.7} = 12\frac{1}{2} \text{ tons per sq. ft.}$$

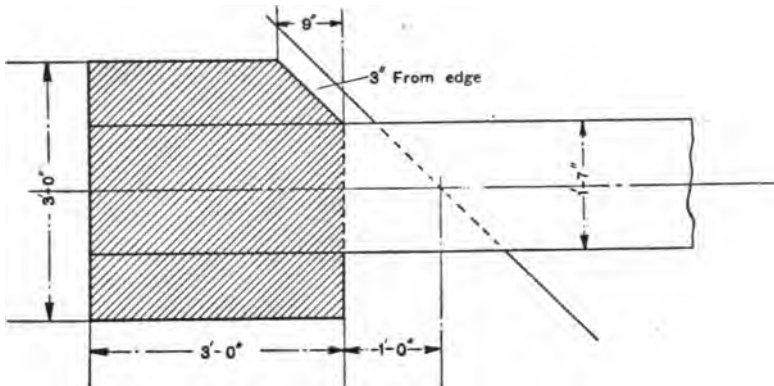


FIG. 130.

Web.—

Maximum end shear = 107 tons.

Working stress = $5\frac{1}{2}$ tons per sq. in.

$$\text{Web area required} = \frac{107}{5\frac{1}{2}} = 19.2 \text{ sq. ins.}$$

$$\text{Depth of web} = (78 - 19.5 \text{ (for holes)}) = 58.5 \text{ sq. ins.}$$

$$\text{Thickness required for shear} = \frac{19.2}{58.5} = 0.33.$$

$$\text{Shearing force per ft. of depth} = \frac{107}{6.5} = 16.4 \text{ tons.}$$

Assume $\frac{1}{2}''$ Web Plate at ends.—Bearing value of $\frac{7}{8}''$ rivet in $\frac{1}{2}''$ plate at 12 tons per sq. in. = $\frac{7}{8} \times \frac{1}{2} \times 12 = 5\frac{1}{4}$ tons.

$$\text{Number of rivets required per ft.} = \frac{16.4}{5.25} = 3.1.$$

$\therefore \frac{1}{2}''$ web with $\frac{7}{8}''$ diameter rivets at $3\frac{3}{4}''$ pitch would suffice, or with 4" pitch (double riveting).

Web Diagrams.—A $\frac{3}{8}$ " web will be sufficient to carry the shearing force after a distance of 14' 0" from the centre of bearing or (say) 15' 0" from the end of the girder.

$\frac{3}{8}$ " web for middle sections of girder. (See Fig. 131.)

Elevation of Girders showing Stiffeners and Cross-bracing Frames.—The construction of these girders is such that they are all similar, i.e. they can all be constructed from the same shop templates, and then one is turned end for end with reference to the other, and given a lead (see Fig. 132) of 5' 0", the girders will then be in their correct positions.

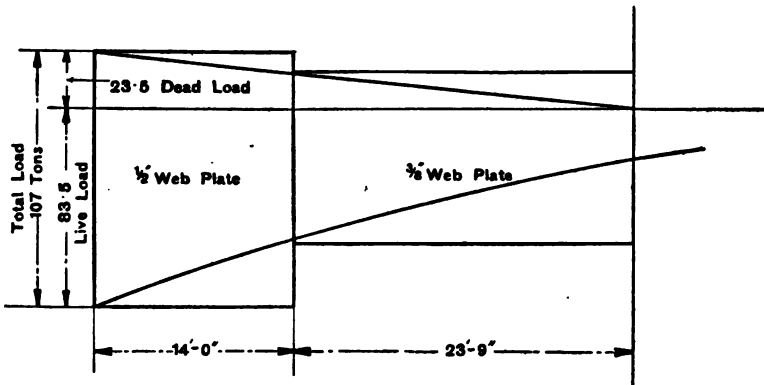


FIG. 131.

Design of Flange and Web Joints.—

Covers for Flange Plates $19" \times \frac{11}{16}"$.

Area of flange plate $19" \times \frac{11}{16}"$ (less 4 holes $\frac{1}{8} \times \frac{5}{8}$) = 10.5 sq. ins.

Load carried at $7\frac{1}{2}$ tons per sq. inch = $10.5 \times 7\frac{1}{2} = 78.7$ tons.

Rivet area required at $5\frac{1}{2}$ tons per sq. inch = $\frac{78.7}{5.5} = 14.3$ sq. ins.

1. $\frac{7}{8}$ " rivet in single shear has an area of 0.6 sq. in.

$\therefore$ Number of rivets required = $\frac{14.3}{0.6} = 24$ rivets.

Make Flange Covers $19" \times \frac{3}{4}"$. (See Fig. 133.)

The rivets in the cover strips are in double shear, but have been considered in single shear in the design.

Angle Covers.—

Area of L Bar $6" \times 6" \times \frac{5}{8}$ (less 3 holes $\frac{1}{8} \times \frac{5}{8}$) = 5.35 sq. ins.

Load carried at $7\frac{1}{2}$ tons per sq. in. = $5.35 \times 7\frac{1}{2} = 40.13$ tons.

Rivet area required at $5\frac{1}{2}$ tons per sq. in. $= \frac{40 \cdot 13}{5\frac{1}{2}} = 7 \cdot 3$ sq. ins.

Number of rivets required $= \frac{7 \cdot 3}{0 \cdot 6} = 12$ rivets.

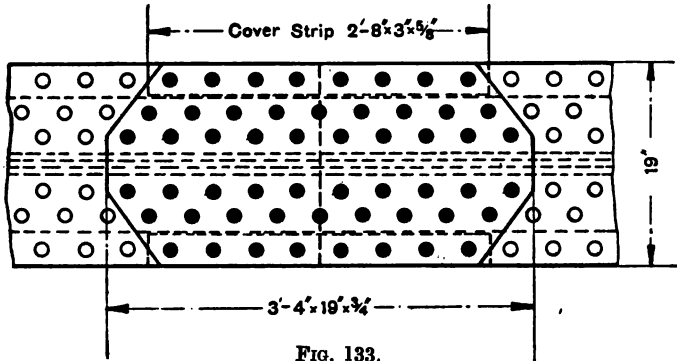


FIG. 133.

The covers will be made from a bent plate $\frac{3}{4}$ " thick, and the joint as shown in Fig. 134.

Double covers will be used, *i.e.* one on each side of the joint. The rivets in the vertical limb of the covers will be in double shear, but have been considered in single shear.

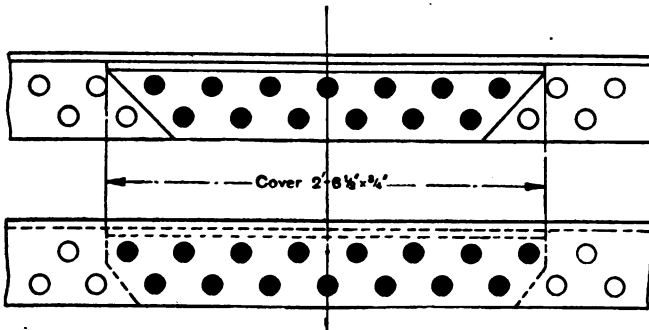


FIG. 134.

The position of the flange plate joints and angle joints are shown in Folding Plate, Fig. 135.

Web Joints.—

Net area of web plate $= (78 \times \frac{1}{2} - 19 \cdot 5 \times \frac{1}{2}) = 29 \cdot 25$ sq. ins.

Number of rivets required in joint $= \frac{29 \cdot 25}{0 \cdot 6}$
 $= 50$ rivets (in single shear).
 $= 25$ „ (in double shear).

For $\frac{3}{8}$ " web joints, number of rivets required = $\frac{3}{8} \times 25 = 10$.

Cover Plates $\frac{3}{8}$ " thick (double covers).

No cover plates are necessary for $\frac{3}{8}$ " web joints where the joint is covered by a tee stiffener, which provides ample rivets, but where the joint occurs at a gusset stiffener double covers $7\frac{1}{2}$ " wide are provided. A stiffener is placed at each web cover.

Position of Flange Plate Joints and Angle Joints.—

2nd tier, theoretical length 54' 0" make 56' 4" actual length

3rd " " " 37' 0" " 39' 8" " " "

(to allow for excess of B.M. at haunches of B.M. diagram.)

The 3rd tier is extended 3' 4" at one end, so as to act as flange cover for joint in 1st tier.

Bottom and top flanges similar, except that 1st tier of top flange has plates 24" wide for floor plate attachments, and is 2' 0" longer than in bottom flange.

The joints in the girders are arranged on the assumption that the girders will be riveted up complete in the shops before being despatched to the site. Girders of this size and weight can easily be transported by rail.

Horizontal wind bracing is hardly necessary in a bridge of this type, where the top connections between the girders are rigid enough to equalise the wind pressure between the two main girders.

Design of Parapet Girders.—As the top of the parapet girder must not

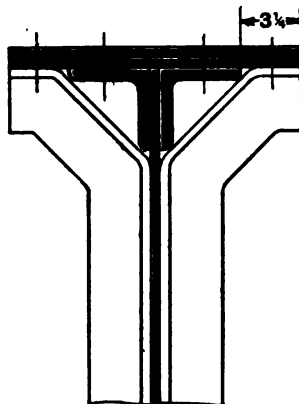


FIG. 136.

be less than 4' 6" above rail level, it must have a minimum depth over angles of 6 ft. If a shallower girder is adopted, a hand-railing with angle-brackets

would be required in addition. To guard against the possibility of a derailment throwing an extra load on the parapet girder, it will be designed to carry one-half of the live load of the main girders, but wind load will be neglected.

Span (c. to c.) of bearings, 75' 6".

	Due to B.M.	Due to S.F.
Live load per girder	= 42 tons.	46½ tons.
Impact allowance (80%)	= 33½ "	37 "
Dead load (steelwork)	= 12 "	12 "
" " (flooring, etc.) = (365 × 75½)	= 12½ "	12½ "
Total	100 "	108 "

$$\text{Maximum B.M. at centre of girder} = \frac{100 \times 75\frac{1}{2}}{8} = 944 \text{ tons-ft.}$$

$$\text{Flange load} = \frac{944}{6} = 158 \text{ tons.}$$

Therefore area required in flanges (at 7½ tons per sq. in. working stress)

$$= \frac{158}{7\frac{1}{2}} = 21.07 \text{ sq. ins.}$$

Area provided

$$\begin{cases} 2 \text{ L } 4\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{8} \text{ (each less 1 hole } \frac{5}{8} \times \frac{1}{8}) = 9.29 \text{ sq. ins.} \\ 2 \text{ Plates } 16'' \times \frac{1}{2}'' \text{ (" " 3 " } \frac{1}{2} \times \frac{1}{8}) = 13.20 \text{ " "} \end{cases}$$

Total 22.49 "

Diagram of Flange Plates.—Fig. 137.

Theoretical length of 2nd tier of plates = 38' 0".

Make tier — 42' 0" long.

Bearing Plates.—

$$\text{Maximum reaction at ends} = \frac{108}{2} = 54 \text{ tons.}$$

$$\text{Bearing area required at 12 tons per sq. ft.} = \frac{54}{12} = 4.5 \text{ sq. ft.}$$

Make bearing plates 3' 0" × 1' 10" × ¾"

Web.—

$$\text{Maximum end shear} = \frac{108}{2} = 54 \text{ tons.}$$

Working stress = 5½ tons per sq. in.

Web area required $= \frac{54}{5\frac{1}{2}} = 10$ sq. ins.

Depth of web $= (72 - 18 \text{ (for holes)}) = 54$ ins.

Thickness required for shear $= \frac{10}{54} = 0.2''$.

Shearing force per ft. of depth $= \frac{54}{6} = 9$ tons.

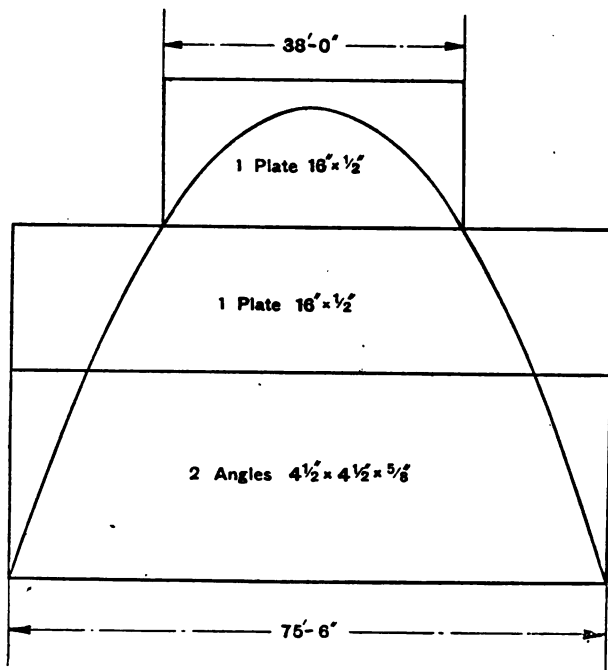


FIG. 137.

Assume a web plate $\frac{3}{8}$ " thick.—Bearing value of 1 $\frac{7}{8}$ " rivet in $\frac{3}{8}$ " plate @ 12 tons per sq. in. $= \frac{7}{8} \times \frac{3}{8} \times 12 = 3.9$ tons.

Number of rivets required per ft. $= \frac{9}{3.9} = 2.4$.

A $\frac{3}{8}$ " web with 4" pitch at ends would suffice.

As $\frac{3}{8}$ " is the minimum thickness advisable for a web plate, the web will be made $\frac{3}{8}$ " thick throughout.

The spacing of stiffeners and position of web plate joints in the Parapet Girders are shown diagrammatically in Fig. 138.

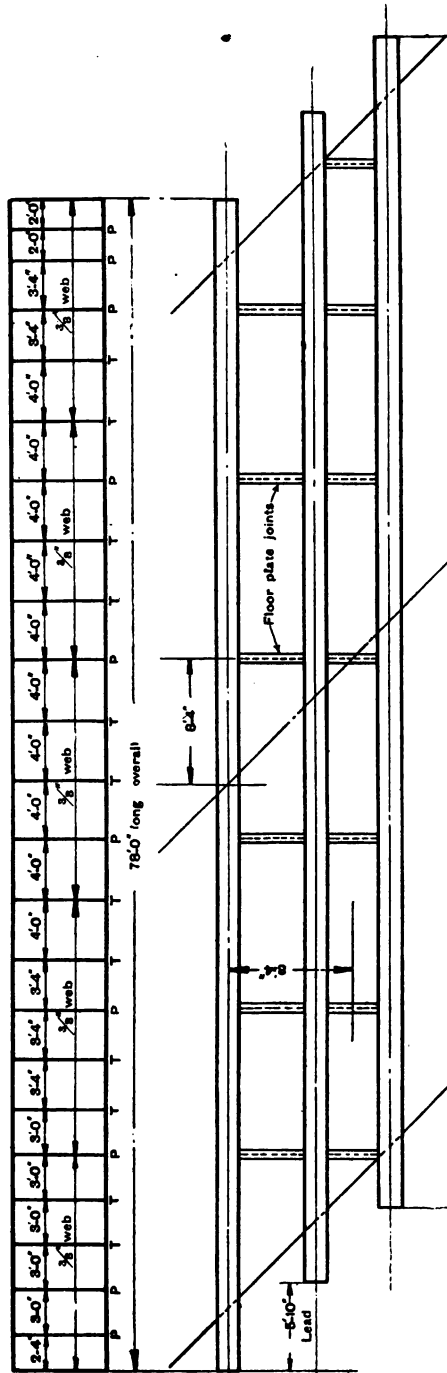


FIG. 138.

Flange Plate Covers.—

Area of plate $16'' \times \frac{1}{2}''$ (less 3 holes $\frac{1}{2}'' \times \frac{5}{8}''$) = 6.6 sq. ins.

Load carried at $7\frac{1}{2}$ tons per sq. in. = $6.6 \times 7\frac{1}{2} = 49.5$ tons.

Rivet area required at $5\frac{1}{2}$ tons per sq. in. = $\frac{49.5}{5.5} = 9$ sq. ins.

Number of $\frac{7}{8}''$ rivets required = $\frac{9}{0.6} = 15$ rivets.

The covers will be $\frac{5}{8}''$ thick $\times$ 3' 0" long as shown in Fig. 139.

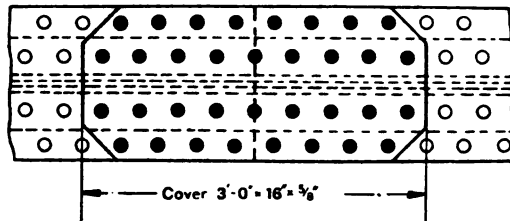


FIG. 139.

Angle Covers.—

Area of angle bar ($4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{5}{8}''$) less 1 hole = 4.57 sq. ins.

Load carried at $7\frac{1}{2}$ tons per sq. in. = $4.57 \times 7\frac{1}{2} = 34$ tons.

Rivet area required at $5\frac{1}{2}$ tons per sq. in. = $\frac{34}{5.5} = 6.2$ sq. ins.

Number of $\frac{7}{8}''$ rivets required = $\frac{6.2}{0.6} = 11$.

The angle covers will be double 4' 0" long, as shown in Fig. 140.

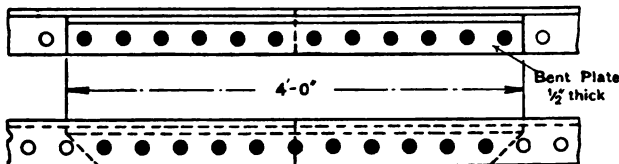


FIG. 140.

The positions of the joints in the flange plates and main angles are shown in Folding Plate, Fig. 141.

The web covers will be similar to those of the main girders.

Waterproofing and Drainage.—The waterproofing and drainage will follow the lines described in Chapter VIII.

Girder Bedstone Levels.—Owing to the combined effect of the main girder camber and the skew of the bridge, in order to avoid bad bracing and

floor connections, it is necessary to set the bedstones at levels which ensure all the girders lying on the same camber curve. These levels can be obtained either by calculation, or by setting out the camber curve to scale and scaling the levels (see Fig. 142).

Assuming the water-level of the river to be 100·00, and the distance from the water-level to rail-level to be 25 ft., the level of the latter will be 125·00.

The construction depth of the bridge at the centre is 8' 7½", made up as follows :—

	ft.	ins.
Top of rail to underside of sleeper	1	1
Ballast and asphalt		7
Depth of girder over flange plates	6	10½
Bottom flange cover		¾
Depth of rivet head		⅝
	8	7½

Deducting this dimension from the rail-level, the level of the underside of the steel work at the centre of the bridge is 116·38, which gives a head-way under the bridge of 16·38 feet.

The bedstone levels for the various girders can now be obtained by reference to the camber diagram, Fig. 142.

Neglecting the effect of camber and assuming all the girders to lie on the same horizontal line, the bedstone level is 115·55, obtained as follows :—

$$\begin{array}{r}
 116\cdot38 \text{ underside of steelwork} \\
 \text{plus } 0\cdot23 \left\{ \begin{array}{l} \text{rivet head, } \frac{5}{8}'' \\ \text{cover plate, } \frac{3}{4}'' \\ \text{2 } 1\frac{1}{8}'' \text{ flanges, } 1\frac{3}{8}'' \end{array} \right. \\
 \hline
 116\cdot61 \\
 \text{less } 1\cdot06 \left\{ \begin{array}{l} \text{bearing plate, } \frac{3}{4}'' \\ \text{rockers, } 1' 0'' \end{array} \right. \\
 \hline
 115\cdot55 \text{ bedstone level.}
 \end{array}$$

Upon reference to the camber diagram, Fig. 142, the difference in levels due to the camber and skew can easily be found.

The bedstone level of girders marked B is 115·55 less the difference in levels of ordinate B and versed sine of camber curve

$$= 115\cdot55 - (0\cdot32 - 0\cdot07) = 115\cdot30$$

$$\text{Bedstone level of girders C} = 115\cdot55 - (0\cdot32 - 0\cdot12) = 115\cdot35$$

$$\text{Bedstone level of girders D} = 115\cdot55 - (0\cdot32 - 0\cdot18) = 115\cdot41$$

$$\text{Bedstone level of girders E} = 115\cdot55 - (0\cdot32 - 0\cdot21) = 115\cdot44$$

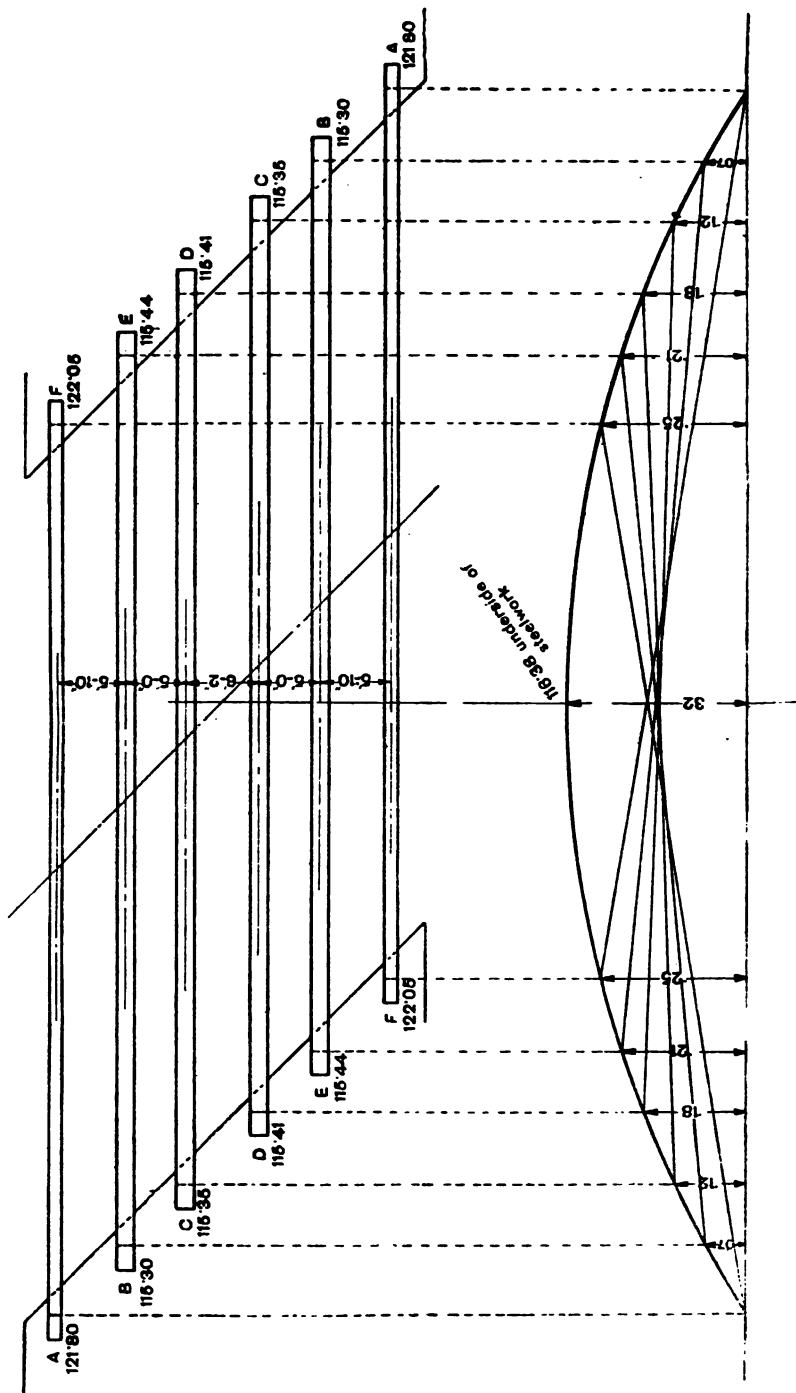


FIG. 142.

It will be noticed that the bedstone levels of the opposite ends of the two outer main girders will be the same, also the levels of the opposite ends of the two inner main girders. The same remark applies to the parapet girders.

Neglecting the effect of camber, the difference in the level of the bedstones of the main girders and parapet girders is $6' 6\frac{7}{8}"$, giving a level of 122.12. The bedstone levels can now be found by reference to the camber diagram.

$$\text{Bedstone level of girders A} = 122.12 - 0.32 = 121.80$$

$$\text{Bedstone level of girders F} = 122.12 - (0.32 - 0.25) = 122.05$$

The design for this bridge is shown on Plate No. II., page 146.

APPENDIX

PROVISIONS OF THE RAILWAYS CLAUSES CONSOLIDATION ACT, 1845, AND THE BOARD OF TRADE REGULATIONS RELATING TO RAILWAY BRIDGES

THE following extracts are from the Railways Clauses Consolidation Act, 1845, and the Board of Trade Regulations for Railways, and must be borne in mind in designing railway bridges.

With road bridges under the railway, the following regulations are obligatory :

A turnpike road bridge must be at least 35 ft. wide in the clear between the abutments, and must have a clear headway of 16 ft. in height for a minimum width of 12 ft.

A public carriage road bridge must be 25 ft. wide in the clear, with a headway of 15 ft. for a minimum width of 10 ft.

A private or occupation road bridge must be 12 ft. wide in the clear, with a headway of 14 ft. for a minimum width of 9 ft.

In skew bridges the dimensions for width must be measured on the square or at right angles to the direction of the road.

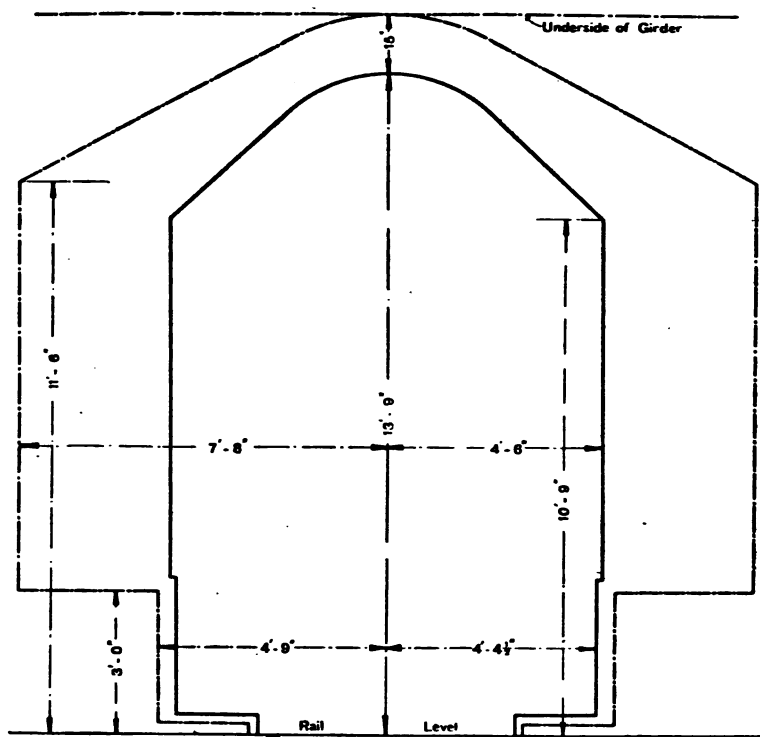
Bridges carried over the railway must have the same width in the clear, measured on the square, between the parapets, as bridges under the railway must have between the abutments.

The amount of headway under these bridges is left to the railway company. The width and height must, of course, be of ample dimensions to accommodate the rolling stock. The ordinary minimum width for a double line of way for present-day rolling stock 4' 8½" gauge, is 26' 6", made up as follows :—

Two lines of way		...	...	...	...	10' 4"
Space between lines of way	...	...	...	...	...	6' 0"
Spaces between rails (outer edge) and abutments						
of bridge	...	...	...	...	2 at 5' 1"	=10' 2"

The ordinary minimum headway is 14 ft., but construction gauges vary on different railways. Fig. 143 shows the construction gauge of one of the leading railways.

In order to carry a road over or under a railway the road has sometimes to be raised or depressed. The gradient of the altered road need not be better than the average gradient of the same road within 250 yds. of the point of crossing, but if the existing gradients within 250 yds. of the point



Lines shown thus ———— refer to minimum fixed structures.

Lines shown thus ———— refer to maximum rolling structures.

FIG. 143.—Structure gauge.

NOTE.—These dimensions are subject to the adjustment due to the super-elevation of the outer rail on curves.

of crossing are better than the gradient of the altered road, then the latter must not be steeper than the following:—

Turnpike roads	...	...	...	...	1 in 30
Public carriage roads	...	...	...	...	1 in 20
Private or occupation roads	...	...	...	...	1 in 16

In cases of bridges carrying roads over a railway, there must be a sufficient parapet on each side of the bridge not less than 4 ft. high, and the

fences on each side of the immediate approaches to the bridge not less than 3 ft. high.

A railway company need not construct public road bridges, either over or under the line, of greater width than that of the road within 50 yds. of the point of crossing, but in no case shall the bridge be narrower than 20 ft. If advantage is taken of this provision, and a bridge constructed of less than the proper width for the particular class of road, the railway company must widen it to the proper width, if at any time subsequent to the construction of the bridge the road be widened.

When stations occur on or near a bridge under the railway, a parapet or fence on each side should be provided, sufficient to prevent passengers falling from the bridge in the dark.

All bridges under the railway should be provided with handrails or parapets, not less than 4 ft. 6 ins. above rail-level, and with projecting platforms for the protection and escape of platelayers.

The steps of footbridges over the lines to be not less than 11 ins. in the tread, nor more than 7 ins. in the rise, and all such staircases to be provided with efficient handrails. Landings must be provided where the height is above 10 ft.

No standing work—other than a passenger platform—shall be nearer to the side of the widest carriage in use on the line than 2 ft. 4 ins. at any point between the level of 2 ft. 6 ins. above the rails and the level of the upper parts of the highest carriage doors. This applies to all arches, abutments, piers, supports, girders, tunnels, bridges, roofs, and other works, and to all projections at the side of a railway constructed to any gauge whatever.

Intervals between adjacent lines of rails are to be not less than 6 ft. for two lines, or lines and sidings, and not less than 9 ft. 6 ins. between additional running lines and main lines.

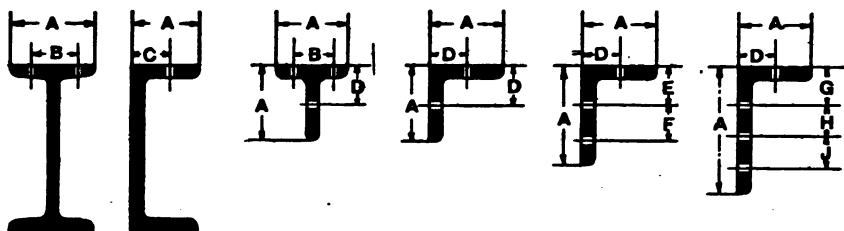
In a wrought-iron or steel bridge the greatest load which can be brought upon it, added to the weight of the superstructure, should not produce a greater strain on any part of the material than 5 tons per sq. in. where wrought iron is used, or $6\frac{1}{2}$ tons per sq. in. where steel is employed.

The heaviest engines, boiler trucks, or travelling cranes in use on railways afford a measure of the greatest moving load to which a bridge can be subjected. These rules apply equally to the main and transverse girders.

All bridges must be designed to resist a wind pressure of 56 lbs. per sq. ft.

The above extracts briefly state the chief requirements of the Railways Clauses Consolidation Act, 1845, and the Board of Trade Regulations affecting railway bridges, but it is most essential that all designers of railway bridges should be quite familiar with the Act and Regulations themselves.

SPACING OF HOLES.



Width in inches. A	Spacing of holes.							
	B	C	D	E	F	G	H	J
2	1	1½	1½					
2½	1½	1½	1½					
3	1½	1½	1½					
3½	1½	2	2					
4	2½	2½	2½					
4½	2½		2½	2	1½			
5	2½		3	2	1½			
5½	3		3½	2½	2			
6	3½		3½	2½	2½	2	1½	1½
6½	3½		4	2½	2½	2	1½	1½
7	4			2½	3	2	1½	1½
7½	4½			2½	3	2	2	2
8	4½			3	3	2	2½	2½
8½				3½	3½	2	2½	2½
9				3½	3½	2	2½	2½
10				3½	4	2½	2½	2½

EXTRAS ON STEEL PLATES, ANGLES, &c.

EXTRAS ON PLATES.

Plates ½ in. thick and up to 1½ in., free of extra. All sketch plates.

Plates, having 9 in. and less of taper (in a straight line), will not be regarded as sketches. Plates over 9 in. taper or requiring more than 4 cuts of the shears will be treated as sketches.

Circular plates, radial plates, and plates sheared to template or to mould, do not rank as sketch plates, but are charged in all cases.

Plates 15 ft. long and upwards under 12 in. broad.

Plates 15 ft. long and upwards 12 in. and under 18 in. broad.

Plates, when guaranteed to stand welding or flanging.

Plates ordered to be mangled. (Makers, however, do not guarantee that the plates will be perfectly flat after mangling.)

Oiling plates ½ in. thick and up.

Extras charged.

EXTRAS ON THIN PLATES.

Under $\frac{3}{8}$ in. to $\frac{5}{16}$ in. thick, inclusive, and 12 $\frac{1}{2}$ lb. plates.

Under $\frac{5}{16}$ in. to $\frac{1}{2}$ in. thick inclusive.

24 in. plates are to be reckoned as $\frac{1}{2}$ in. plates.

10 lb. plates.

Under $\frac{1}{2}$ in. to $\frac{3}{8}$ in. thick inclusive.

7 $\frac{1}{2}$ lb. plates.

No plates under $\frac{3}{8}$ in. thick will be supplied free of extra.

Extras charged.

EXTRAS FOR WIDTHS ON PLATES.

For every 3 inches or part over width indicated in this Table extra charged.											
Thickness of plate	$\frac{5}{16}$ " and under $\frac{3}{8}$ "	$\frac{3}{8}$ " and under $\frac{7}{16}$ "	$\frac{7}{16}$ " and under $\frac{1}{2}$ "	$\frac{1}{2}$ " and under $\frac{3}{4}$ "	$\frac{3}{4}$ " and under $\frac{5}{8}$ "	$\frac{5}{8}$ " and under $\frac{3}{2}$ "	$\frac{3}{2}$ " and under $\frac{7}{4}$ "	$\frac{7}{4}$ " and under 1"	1" and under 1 $\frac{1}{4}$ "	1 $\frac{1}{4}$ " and under 1 $\frac{1}{2}$ "	1 $\frac{1}{2}$ " and thicker
Width supplied free of extra	66"	75"	84"	90"	96"	96"	96"	96"	96"	96"	96"

EXTRAS ON ANGLES.

6 to 11 united inches (including 6 in. $\times$ 6 in.) $\times$ $\frac{3}{4}$ in. and up.

Under 6 united inches, per inch, or part.

Over 11 united inches, per inch or part (excluding 6 in. $\times$ 6 in.)

On angles 6 united inches of section and larger, an extra will be charged for thinness under $\frac{3}{8}$ in. to $\frac{5}{16}$ in.

Under $\frac{5}{16}$ in. to $\frac{1}{2}$ in.

On all angles under $\frac{1}{2}$ in. to $\frac{3}{8}$ in. thick, inclusive.

On all angles under $\frac{3}{8}$ in. to $\frac{1}{2}$ in. thick, inclusive.

Round backed angles—extra for section.

Basis price.

Extras charged over Basis Price.

EXTRAS ON ZED BARS.

Extra for section over the basis price for angles.

Under 10 and over 14 united inches, per half-inch or part.

Do.

EXTRAS ON CHANNELS.

Extra for section over the basis price for angles 7 in. $\times$ 3 in. $\times$ $\frac{1}{2}$ in. to 10 in. $\times$ 3 $\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in., free of extra for size.

7 in. $\times$ 3 in. to 10 in. $\times$ 3 $\frac{1}{2}$ in. under $\frac{1}{2}$ in. to $\frac{7}{16}$ in.

7 in. $\times$ 3 in. to 10 in. $\times$ 3 $\frac{1}{2}$ in. under $\frac{7}{16}$ in. to $\frac{3}{4}$ in. (Thicknesses to be read on the web only.)

6 in., 11 in., and 12 in. sections, with not less than 3 in. flange.

6 in. $\times$ 2 $\frac{1}{2}$ in. channels.

Sections under 6 in. web, subject to arrangement.

Do.

EXTRAS ON T BARS.

Extra for section over the basis price for angles.

Extra for size over 10 united inches per half-inch or part.

Under 6 united inches, per inch or part.

Extras for thinness, same as angles.

EXTRAS ON FLATS.

Extras for section over the basis price for angles :—

Ordinary flats, $1\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in. and up.

Wide flats 8 in. and up $\times$ $\frac{1}{2}$ in. and up

Extras for size : 1 in. and $1\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in.

$\frac{7}{8}$ in. $\times$ $\frac{1}{4}$ in.

Under $\frac{1}{4}$ in. and not less than $\frac{3}{16}$ in. thick.

Under $\frac{3}{16}$ in. thick.

Ordinary cut lengths of angles, flats, etc., 5 feet and over 3 feet, extra. 3 feet to 2 feet extra. Cold straightening extra.

Special extras will be charged for cutting to short lengths, or to exact lengths, and for special straightening.

MAXIMUM DIMENSIONS TO WHICH PLATES ARE ROLLED.

(See Explanatory Note below.)

Thickness in inches.	Length in feet and inches.	Breadth in inches.	Area in square feet.
ins.	ft.	ins.	sq. ft.
$\frac{1}{4}$	30·0	72	110
$\frac{5}{16}$	34·0	80	135
$\frac{3}{8}$	36·0	102	180
$\frac{7}{16}$	40·0	108	210
$\frac{1}{2}$	48·0	114	230
$\frac{9}{16}$	50·0	118	240
$\frac{5}{8}$	50·0	120	250
$\frac{11}{16}$	50·0	130	260
$\frac{3}{4}$	50·0	138	270
$\frac{7}{8}$	50·0	140	300
1	50·0	140	300

Plates of larger dimensions than those shown above may be undertaken by special arrangement.

NOTE.—It will be understood that a plate or sheet cannot be rolled to both maximum LENGTH and WIDTH, but only to the maximum AREA; therefore the maximum area given above, divided by a length in feet (not exceeding maximum) will give maximum width for that length in feet—to which the plate can be rolled.

WEIGHT.—The weight of a cubic foot of Siemens' Steel is, as nearly as possible, 490 lbs., a square foot 1 in. thick, weighs, therefore, 40·8 lbs., and this should be taken as a basis when calculating the weight of steel plates, angles, bars, etc. It is impossible always to roll exactly to weight, and a deviation of at least 5 per cent. should be allowed.

TABLE.

WEIGHTS OF MISCELLANEOUS SUBSTANCES.

Asphalt	...	...	...	...	...	140-156 lb.	per cube foot.
Brickwork, pressed	...	...	...	...	...	130-150	" "
Brickwork, ordinary	...	...	...	...	...	110-130	" "
Brickwork, soft	...	...	...	...	...	90-100	" "
Cement (Portland)	...	...	...	...	...	86-100	" "
Concrete	...	...	...	...	...	120-140	" "
Concrete, reinforced	...	...	...	...	...	150-160	" "
Granite	...	...	...	...	...	140-180	" "
Limestone (average)	...	...	...	...	...	163	" "
Sandstone (average)	...	...	...	...	...	153	" "
Timber	...	...	...	...	...	45	" "
Wrought iron	...	...	...	...	...	480	" "
Cast iron	...	...	...	...	...	450	" "
Mild steel	...	...	...	...	...	490	" "
Cast steel	...	...	...	...	...	492	" "
Brass	...	...	...	...	...	530	" "
Lead	...	...	...	...	...	710	" "
Gunmetal	...	...	...	...	...	530	" "
Ballast	...	...	...	...	...	120	" "
Macadamised road	...	...	...	...	...	120	" "
Permanent way for a single line(exclusive of ballast)	...	...	...	...	...	175	per lineal foot.
Gas tubing: $\frac{3}{4}$ in. bore	...	...	...	...	...	1.18	" "
" 1 in. "	...	...	...	...	...	1.79	" "
" $1\frac{1}{4}$ in. "	...	...	...	...	...	2.52	" "
" $1\frac{1}{2}$ in. "	...	...	...	...	...	2.97	" "

PLATE GIRDER BRIDGES

WEIGHT OF FLAT ROLLED STEEL IN LBS. PER LINEAL FOOT.

Width in inches.	Thickness in inches.							
	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
... .. 1	213	425	638	850	106	128	149	170
... 1 $\frac{1}{4}$...	266	531	797	106	133	159	186	213
1 $\frac{1}{4}$	319	638	956	128	159	191	223	255
... 1 $\frac{1}{2}$...	372	744	112	149	186	223	260	298
... .. 2	425	850	128	170	213	255	298	340
... 2 $\frac{1}{4}$...	478	956	143	191	239	287	335	383
2 $\frac{1}{4}$	531	106	159	213	266	319	372	425
... 2 $\frac{1}{2}$...	584	117	175	234	292	351	409	468
... .. 3	638	128	191	255	319	383	446	510
... 3 $\frac{1}{4}$...	691	138	207	276	345	414	483	553
3 $\frac{1}{4}$	744	149	223	298	372	446	521	595
... 3 $\frac{1}{2}$...	797	159	239	319	398	478	558	638
... .. 4	850	170	255	340	425	510	595	690
... 4 $\frac{1}{4}$...	903	181	271	361	452	542	632	723
4 $\frac{1}{4}$	956	191	287	383	478	574	669	765
... 4 $\frac{1}{2}$...	101	202	303	404	505	606	707	808
... .. 5	106	213	319	425	531	638	744	850
... 5 $\frac{1}{4}$...	112	223	335	446	558	669	781	893
5 $\frac{1}{4}$	117	234	351	468	584	701	818	935
... 5 $\frac{1}{2}$...	122	244	367	489	611	733	855	978
... .. 6	128	255	383	510	638	765	893	1020
... 6 $\frac{1}{4}$...	133	266	398	531	664	797	930	1063
6 $\frac{1}{4}$	138	276	414	553	691	829	967	1105
... 6 $\frac{1}{2}$...	143	287	430	574	717	861	1004	1148
... .. 7	149	298	446	595	744	893	1041	1190
... 7 $\frac{1}{4}$...	154	308	462	616	770	924	1078	1233
7 $\frac{1}{4}$	159	319	478	638	797	956	1116	1275
... 7 $\frac{1}{2}$...	165	329	494	659	823	988	1153	1318
... .. 8	170	340	510	680	850	1020	1190	1360
... 8 $\frac{1}{4}$...	175	351	526	701	877	1052	1227	1403
8 $\frac{1}{4}$	181	361	542	723	903	1084	1264	1445
... 8 $\frac{1}{2}$...	186	372	558	744	930	1116	1302	1488
... .. 9	191	383	574	765	956	1148	1339	1530
... 9 $\frac{1}{4}$...	197	393	590	786	983	1180	1376	1573
9 $\frac{1}{4}$	202	404	606	808	1009	1211	1413	1615
... 9 $\frac{1}{2}$...	207	414	622	829	1036	1243	1450	1658
... .. 10	213	425	638	850	1063	1275	1488	1700
... 10 $\frac{1}{4}$...	218	436	653	871	1089	1307	1525	1743
10 $\frac{1}{4}$	223	446	670	893	1116	1339	1562	1785
... 10 $\frac{1}{2}$...	228	457	685	914	1142	1371	1599	1828
... .. 11	234	468	701	935	1169	1403	1636	1870
... 11 $\frac{1}{4}$...	239	478	717	956	1195	1434	1673	1913
11 $\frac{1}{4}$	244	489	733	978	1222	1466	1711	1955
... 11 $\frac{1}{2}$...	250	500	749	999	1248	1498	1748	1998
... .. 12	255	510	765	1020	1275	1530	1785	2040

WEIGHT OF FLAT ROLLED STEEL IN LBS. PER LINEAL FOOT.

Thickness in inches.								Width in inches.	
$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1		
1-91	2-13	2-34	2-55	2-76	2-98	3-19	3-40	1	...
2-39	2-66	2-92	3-19	3-45	3-72	3-98	4-25	...	1½
2-87	3-19	3-51	3-83	4-14	4-46	4-78	5-10	...	1½
3-35	3-72	4-09	4-46	4-83	5-21	5-58	5-95	...	1½
3-83	4-25	4-68	5-10	5-53	5-95	6-38	6-80	2	...
4-30	4-78	5-26	5-74	6-22	6-69	7-17	7-65	...	2½
4-78	5-31	5-84	6-38	6-91	7-44	7-97	8-50	...	2½
5-26	5-84	6-43	7-01	7-60	8-18	8-77	9-35	...	2½
5-74	6-38	7-01	7-65	8-29	8-93	9-56	10-20	3	...
6-22	6-91	7-60	8-29	8-98	9-67	10-36	11-05	...	3½
6-70	7-44	8-18	8-93	9-67	10-41	11-16	11-90	...	3½
7-17	7-97	8-77	9-56	10-36	11-16	11-95	12-75	...	3½
7-65	8-50	9-35	10-20	11-05	11-90	12-75	13-60	4	...
8-13	9-03	9-93	10-84	11-74	12-64	13-55	14-45	...	4½
8-61	9-56	10-52	11-48	12-43	13-39	14-34	15-30	...	4½
9-08	10-09	11-10	12-11	13-12	14-13	15-14	16-15	...	4½
9-56	10-63	11-69	12-75	13-81	14-88	15-94	17-00	5	...
10-04	11-16	12-27	13-39	14-50	15-62	16-73	17-85	...	5½
10-52	11-69	12-86	14-03	15-19	16-36	17-53	18-70	...	5½
11-00	12-22	13-44	14-66	15-88	17-11	18-33	19-55	...	5½
11-48	12-75	14-03	15-30	16-58	17-85	19-13	20-40	6	...
11-95	13-28	14-61	15-94	17-27	18-59	19-92	21-25	...	6½
12-43	13-81	15-19	16-58	17-96	19-34	20-72	22-10	...	6½
12-91	14-34	15-78	17-21	18-65	20-08	21-52	22-95	...	6½
13-39	14-88	16-36	17-85	19-34	20-83	22-31	23-80	7	...
13-87	15-41	16-95	18-49	20-03	21-57	23-11	24-65	...	7½
14-34	15-94	17-53	19-13	20-72	22-31	23-91	25-50	...	7½
14-82	16-47	18-12	19-76	21-41	23-06	24-70	26-35	...	7½
15-30	17-00	18-70	20-40	22-10	23-80	25-50	27-20	8	...
15-78	17-53	19-28	21-04	22-79	24-54	26-30	28-05	...	8½
16-26	18-06	19-87	21-68	23-48	25-29	27-10	28-90	...	8½
16-73	18-59	20-45	22-31	24-17	26-03	27-89	29-75	...	8½
17-21	19-13	21-04	22-95	24-86	26-78	28-69	30-60	9	...
17-69	19-66	21-62	23-59	25-55	27-52	29-48	31-45	...	9½
18-17	20-19	22-21	24-23	26-24	28-26	30-28	32-30	...	9½
18-65	20-72	22-79	24-86	26-93	29-01	31-08	33-15	...	9½
19-13	21-25	23-38	25-50	27-63	29-75	31-88	34-00	10	...
19-60	21-78	23-96	26-14	28-32	30-49	32-67	34-85	...	10½
20-08	22-31	24-54	26-78	29-01	31-24	33-47	35-70	...	10½
20-56	22-84	25-13	27-41	29-70	31-98	34-27	36-55	...	10½
21-04	23-38	25-71	28-05	30-39	32-73	35-06	37-40	11	...
21-52	23-91	26-30	28-69	31-08	33-47	35-86	38-25	...	11½
22-00	24-44	26-88	29-33	31-77	34-21	36-66	39-10	...	11½
22-47	24-97	27-47	29-96	32-46	34-96	37-45	39-95	...	11½
22-95	25-50	28-05	30-60	33-15	35-70	38-25	40-80	12	...

WEIGHT OF FLAT ROLLED STEEL IN LBS. PER LINEAL FOOT.

Width in inches.	Thickness in inches.							
	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
13	2.76	5.53	8.29	11.05	13.81	16.58	19.34	22.10
14	2.98	5.95	8.93	11.90	14.88	17.85	20.83	23.80
15	3.19	6.38	9.56	12.75	15.94	19.13	22.31	25.50
16	3.40	6.80	10.20	13.60	17.00	20.40	23.80	27.20
17	3.61	7.23	10.84	14.45	18.06	21.68	25.29	28.90
18	3.83	7.65	11.48	15.30	19.13	22.95	26.78	30.60
19	4.04	8.08	12.11	16.15	20.19	24.23	28.26	32.30
20	4.25	8.50	12.75	17.00	21.25	25.50	29.75	34.00
21	4.46	8.93	13.39	17.85	22.31	26.78	31.24	35.70
22	4.68	9.35	14.03	18.70	23.38	28.05	32.72	37.40
23	4.89	9.78	14.66	19.55	24.44	29.33	34.21	39.10
24	5.10	10.20	15.30	20.40	25.50	30.60	35.70	40.80
25	5.31	10.63	15.94	21.25	26.56	31.88	37.19	42.50
26	5.53	11.05	16.58	22.10	27.63	33.15	38.68	44.20
27	5.74	11.48	17.21	22.95	28.69	34.43	40.16	45.90
28	5.95	11.90	17.85	23.80	29.75	35.70	41.65	47.60
29	6.16	12.33	18.49	24.65	30.81	36.98	43.14	49.30
30	6.38	12.75	19.13	25.50	31.88	38.25	44.63	51.00
31	6.59	13.18	19.76	26.35	32.94	39.53	46.11	52.70
32	6.80	13.60	20.40	27.20	34.00	40.80	47.60	54.40
33	7.01	14.03	21.04	28.05	35.06	42.08	49.09	56.10
34	7.23	14.45	21.68	28.90	36.13	43.35	50.58	57.80
35	7.44	14.88	22.31	29.75	37.19	44.63	52.06	59.50
36	7.65	15.30	22.95	30.60	38.25	45.90	53.55	61.20
37	7.86	15.73	23.59	31.45	39.31	47.18	55.04	62.90
38	8.08	16.15	24.23	32.30	40.38	48.45	56.53	64.60
39	8.29	16.58	24.86	33.15	41.44	49.73	58.01	66.30
40	8.50	17.00	25.50	34.00	42.50	51.00	59.50	68.00
41	8.71	17.43	26.14	34.85	43.56	52.28	60.99	69.70
42	8.93	17.85	26.78	35.70	44.63	53.55	62.48	71.40
43	9.14	18.28	27.41	36.55	45.69	54.83	63.96	73.10
44	9.35	18.70	28.05	37.40	46.75	56.10	65.45	74.80
45	9.56	19.13	28.69	38.25	47.81	57.38	66.94	76.50
46	9.78	19.55	29.33	39.10	48.88	58.65	68.43	78.20
47	9.99	19.98	29.96	39.95	49.94	59.93	69.91	79.90
48	10.20	20.40	30.60	40.80	51.00	61.20	71.40	81.60
50	10.63	21.25	31.88	42.50	53.13	63.75	74.38	85.00
52	11.05	22.10	33.15	44.20	55.25	66.30	77.35	88.40
54	11.48	22.95	34.43	45.90	57.38	68.85	80.33	91.80
VALUES FOR ADDITIONAL WIDTHS OF $\frac{1}{8}$ ", $\frac{1}{4}$ " AND $\frac{3}{8}$ ".								
$\frac{1}{8}$	.053	.106	.159	.213	.266	.319	.372	.425
... $\frac{1}{4}$...	.106	.213	.319	.425	.531	.638	.744	.850
... .. $\frac{3}{8}$	.159	.319	.478	.638	.797	.956	1.116	1.275

APPENDIX

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WEIGHT OF FLAT ROLLED STEEL IN LBS. PER LINEAL FOOT.

Thickness in inches.								Width in inches.
$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	1	
24.86	27.63	30.39	33.15	35.91	38.68	41.44	44.20	13
26.78	29.75	32.73	35.70	38.66	41.65	44.63	47.60	14
28.69	31.88	35.06	38.25	41.44	44.63	47.81	51.00	15
30.60	34.00	37.40	40.80	44.20	47.60	51.00	54.40	16
32.51	36.13	39.74	43.35	46.96	50.58	54.19	57.80	17
34.43	38.25	42.08	45.90	49.73	53.55	57.38	61.20	18
36.34	40.38	44.41	48.45	52.49	56.53	60.56	64.60	19
38.25	42.50	46.75	51.00	55.25	59.50	63.75	68.00	20
40.16	44.63	49.09	53.55	58.01	62.48	66.94	71.40	21
42.08	46.75	51.43	56.10	60.78	65.45	70.13	74.80	22
43.99	48.88	53.76	58.65	63.54	68.43	73.31	78.20	23
45.90	51.00	56.10	61.20	66.30	71.40	76.50	81.60	24
47.81	53.13	58.44	63.75	69.06	74.38	79.69	85.00	25
49.73	55.25	60.78	66.30	71.83	77.35	82.88	88.40	26
51.64	57.38	63.11	68.85	74.59	80.33	86.06	91.80	27
53.55	59.50	65.45	71.40	77.35	83.30	89.25	95.20	28
55.46	61.63	67.79	73.95	80.11	86.28	92.44	98.60	29
57.38	63.75	70.13	76.50	82.88	89.25	95.63	102.00	30
59.29	65.88	72.46	79.05	85.64	92.23	98.81	105.40	31
61.20	68.00	74.80	81.60	88.40	95.20	102.00	108.80	32
63.11	70.13	77.14	84.15	91.16	98.18	105.19	112.20	33
65.03	72.25	79.48	86.70	93.93	101.15	108.38	115.60	34
66.94	74.38	81.81	89.25	96.69	104.13	111.56	119.00	35
68.85	76.50	84.15	91.80	99.45	107.10	114.75	122.40	36
70.76	78.63	86.49	94.35	102.21	110.08	117.94	125.80	37
72.68	80.75	88.83	96.90	104.98	113.05	121.13	129.20	38
74.59	82.88	91.16	99.45	107.74	116.03	124.31	132.60	39
76.50	85.00	93.50	102.00	110.50	119.00	127.50	136.00	40
78.41	87.13	95.84	104.55	113.26	121.98	130.69	139.40	41
80.33	89.25	98.18	107.10	116.03	124.95	133.88	142.80	42
82.24	91.38	100.51	109.65	118.79	127.93	137.06	146.20	43
84.15	93.50	102.85	112.20	121.55	130.90	140.25	149.60	44
86.06	95.63	105.19	114.75	124.31	133.88	143.44	153.00	45
87.98	97.75	107.53	117.30	127.08	136.85	146.63	156.40	46
89.89	99.88	109.86	119.85	129.84	139.83	149.81	159.80	47
91.80	102.00	112.20	122.40	132.60	142.80	153.00	163.20	48
95.63	106.25	116.88	127.50	138.13	148.75	159.38	170.00	50
99.45	110.50	121.55	132.60	143.65	154.70	165.75	176.80	52
103.28	114.75	126.23	137.70	149.18	160.65	172.13	183.60	54
VALUES FOR ADDITIONAL WIDTHS OF $\frac{1}{4}$ ", $\frac{1}{2}$ " AND $\frac{3}{4}$ ".								
.478	.531	.584	.638	.691	.744	.797	.850	 $\frac{1}{4}$
.956	1.063	1.169	1.275	1.381	1.488	1.594	1.700	 $\frac{1}{2}$
1.434	1.594	1.753	1.913	2.072	2.231	2.391	2.550	$\frac{3}{4}$

SECTIONAL AREAS OF ANGLES IN SQUARE INCHES.

Breadths of flanges added in inches.	Thickness of angle in inches.											
	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
$2\frac{1}{2}$...	.43	.56										
... 3	.53	.69	.84									
$3\frac{1}{2}$...	.62	.81	1.00									
... 4	.72	.94	1.15	1.36								
$4\frac{1}{2}$...	.81	1.06	1.31	1.55	1.78							
... 5		1.19	1.46	1.73	2.00	2.25						
$5\frac{1}{2}$...		1.31	1.62	1.92	2.21	2.50	2.78					
... 6		1.44	1.78	2.11	2.44	2.75	3.06	3.36				
$6\frac{1}{2}$...			1.93	2.30	2.65	3.00	3.34	3.67				
... 7			2.09	2.49	2.87	3.25	3.62	3.99	4.34			
$7\frac{1}{2}$...			2.25	2.67	3.09	3.50	3.90	4.30	4.68			
... 8			2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44		
$8\frac{1}{2}$...				3.05	3.53	4.00	4.47	4.93	5.38	5.82		
... 9				3.24	3.75	4.25	4.75	5.24	5.72	6.19	7.11	
$9\frac{1}{2}$...				3.42	3.97	4.50	5.03	5.55	6.06	6.56	7.55	
... 10				3.61	4.18	4.75	5.31	5.86	6.40	6.94	7.99	9.00
$10\frac{1}{2}$...					4.40	5.00	5.59	6.17	6.75	7.31	8.42	9.50
... 11					4.62	5.25	5.87	6.48	7.09	7.69	8.86	10.00
$11\frac{1}{2}$...					4.84	5.50	6.15	6.80	7.43	8.06	9.30	10.50
... 12					5.06	5.75	6.44	7.11	7.78	8.44	9.74	11.00
... 16						7.75	8.68	9.61	10.53	11.44	13.23	15.00

WEIGHTS OF STEEL ANGLES IN LBS. PER LINEAL FOOT.

Breadths of flanges added in inches.	Thickness of angle in inches.											
	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
$2\frac{1}{2}$...	1.47	1.91										
... 3	1.79	2.33	2.85									
$3\frac{1}{2}$...	2.11	2.76	3.39	3.98								
... 4	2.43	3.19	3.92	4.62								
$4\frac{1}{2}$...	2.75	3.61	4.45	5.26	6.05							
... 5		4.04	4.98	5.89	6.78	7.65						
$5\frac{1}{2}$...		4.46	5.51	6.53	7.53	8.50	9.44	10.36				
... 6		4.90	6.05	7.18	8.28	9.36	10.41	11.43				
$6\frac{1}{2}$...			6.58	7.81	9.02	10.20	11.36	12.49				
... 7			7.11	8.45	9.76	11.05	12.31	13.55	14.76			
$7\frac{1}{2}$...			7.64	9.08	10.50	11.90	13.27	14.61	15.92			
... 8			8.17	9.72	11.25	12.75	14.22	15.67	17.09	18.49		
$8\frac{1}{2}$...				10.37	12.01	13.61	15.19	16.74	18.27	19.77		
... 9				11.00	12.74	14.46	16.14	17.80	19.44	21.04	24.18	
$9\frac{1}{2}$...				11.64	13.49	15.31	17.10	18.87	20.61	22.32	25.67	
... 10				12.27	14.23	16.15	18.05	19.92	21.77	23.59	27.15	30.60
$10\frac{1}{2}$...					14.97	17.00	19.01	20.98	22.94	24.86	28.63	32.30
... 11					15.70	17.84	19.95	22.04	24.10	26.13	30.11	34.00
$11\frac{1}{2}$...					16.46	18.70	20.92	23.11	25.27	27.41	31.61	35.70
... 12					17.21	19.56	21.89	24.18	26.45	28.70	33.11	37.41
... 16						26.35	29.52	32.67	35.79	38.89	45.00	51.00

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